VOL. XXXVI. No. 10. OCTOBER, 1958

THE STRUCTURAL ENGINEER

THE JOURNAL OF THE
INSTITUTION OF STRUCTURAL ENGINEERS

OCT 22 1958

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The President 1958—1959

Aluminium and Tubular Steel as Applied to the Construction of Some Recent Aircraft Hangars

by L. E. Ward (Member)

The Use of Arc Welding in Structural Engineering

by J. S. Allen

Volume Changes in Soils

by E. Ll. Morgan (Associate-Member)

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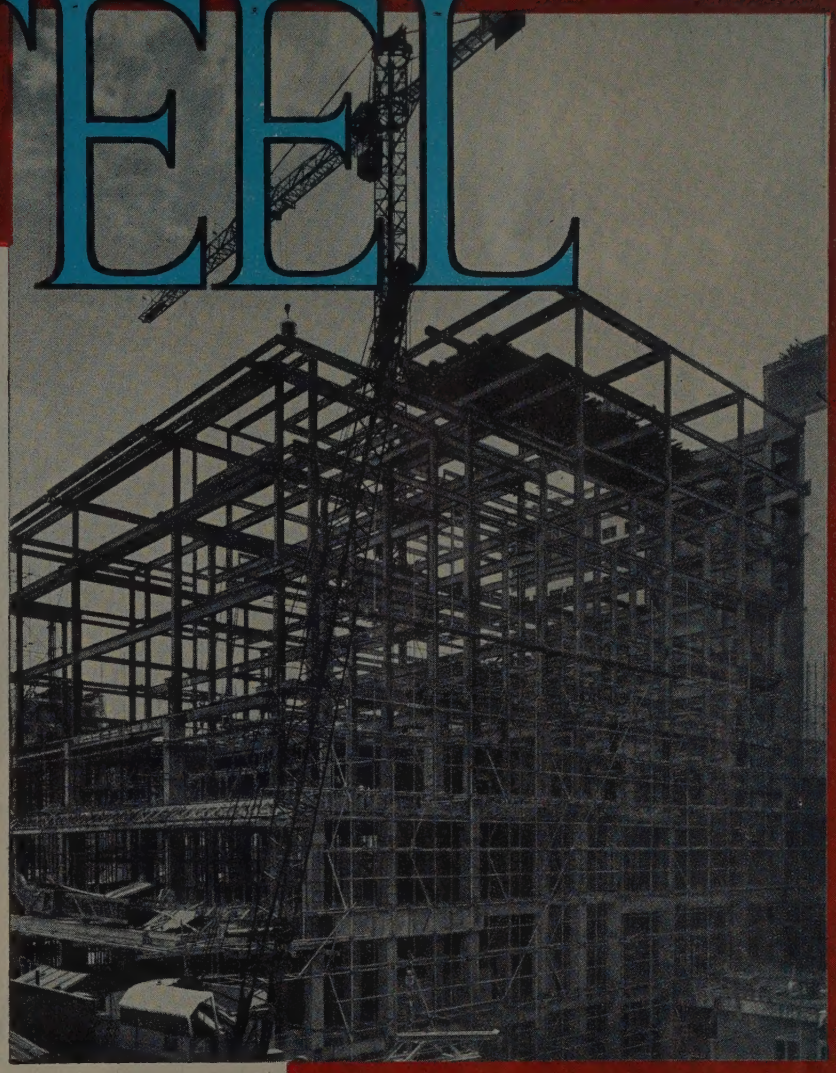

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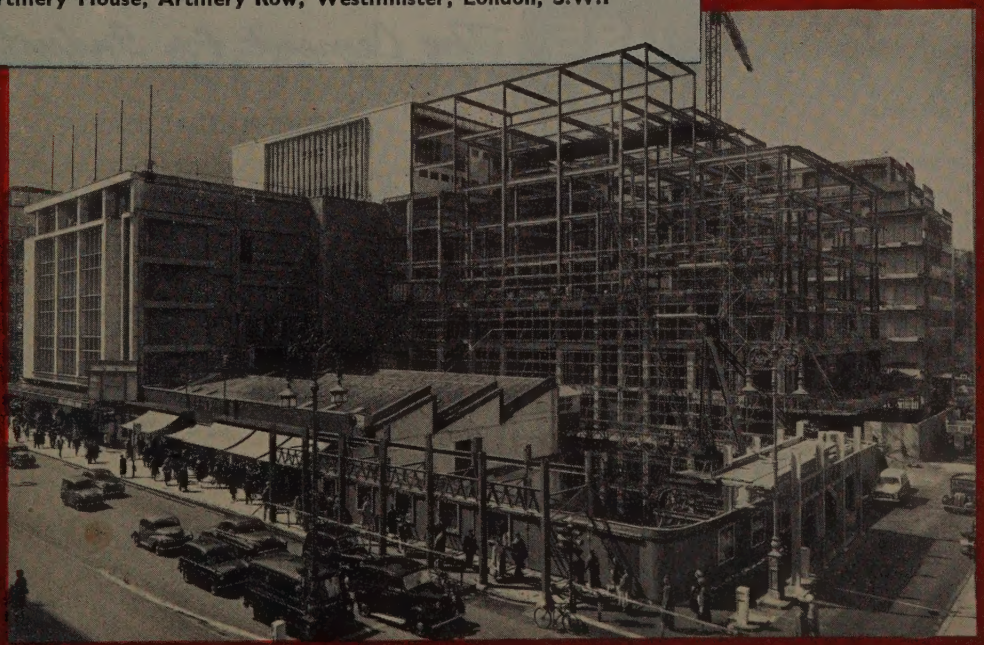

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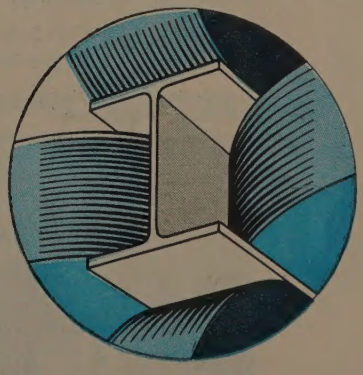
The range of British Standard beams, channels and angles continues to be available.

Diagram showing the arrangement of the rolls, which are adjustable to control the flange and web thickness. The adjustment does not appreciably alter the surfaces shown in blue in the upper illustration.



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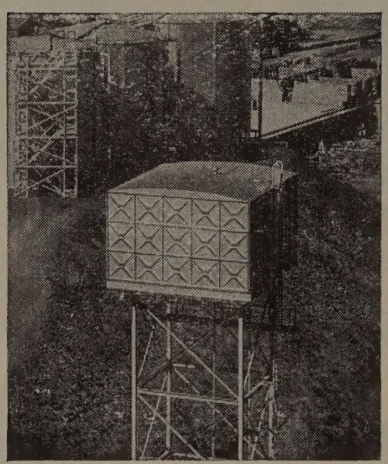
PRESSED STEEL TANKS

Storage for 19,100 gallons at an Iron and Steel Works, England



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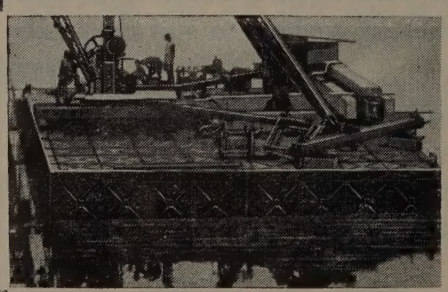
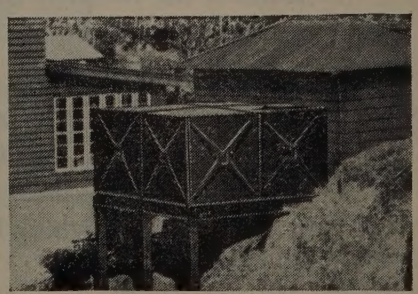
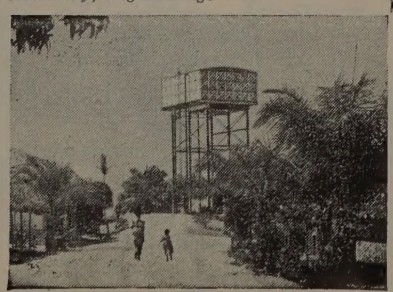


A pontoon for civil engineering work made from units built with pressed steel tank plates



Storage for 1,400 gallons at a tea garden, Ceylon

Storage for 65,100 gallons for a rural community, Belgian Congo



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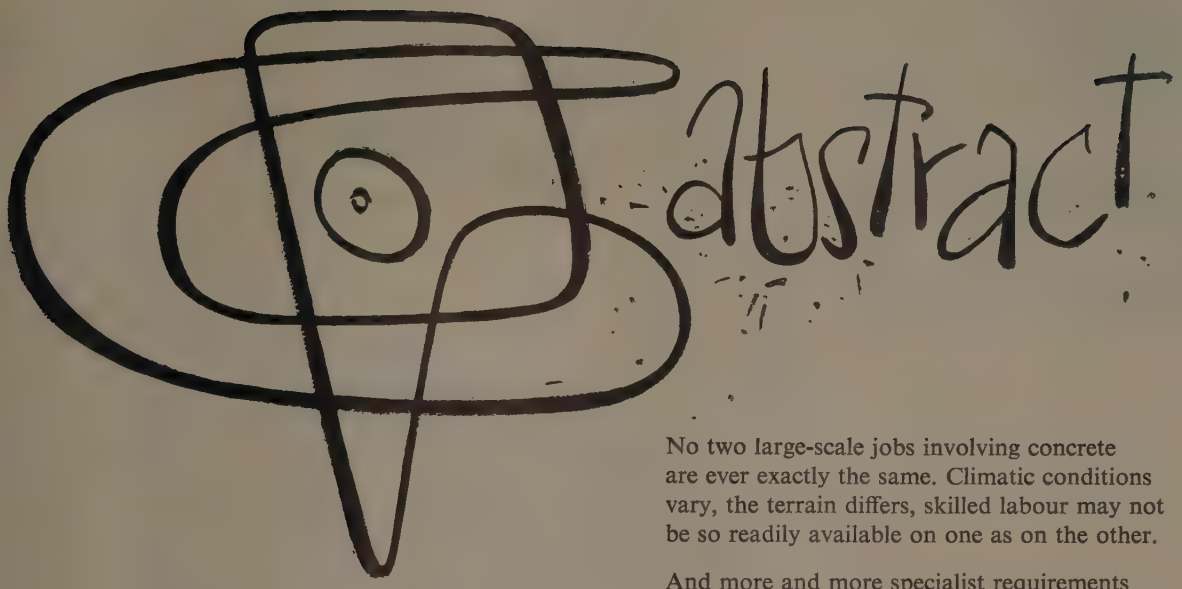

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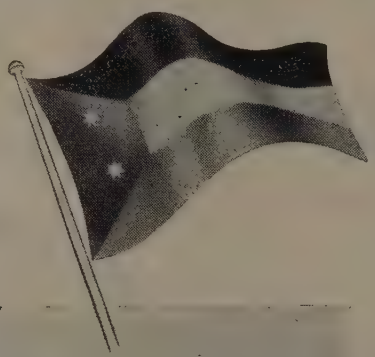

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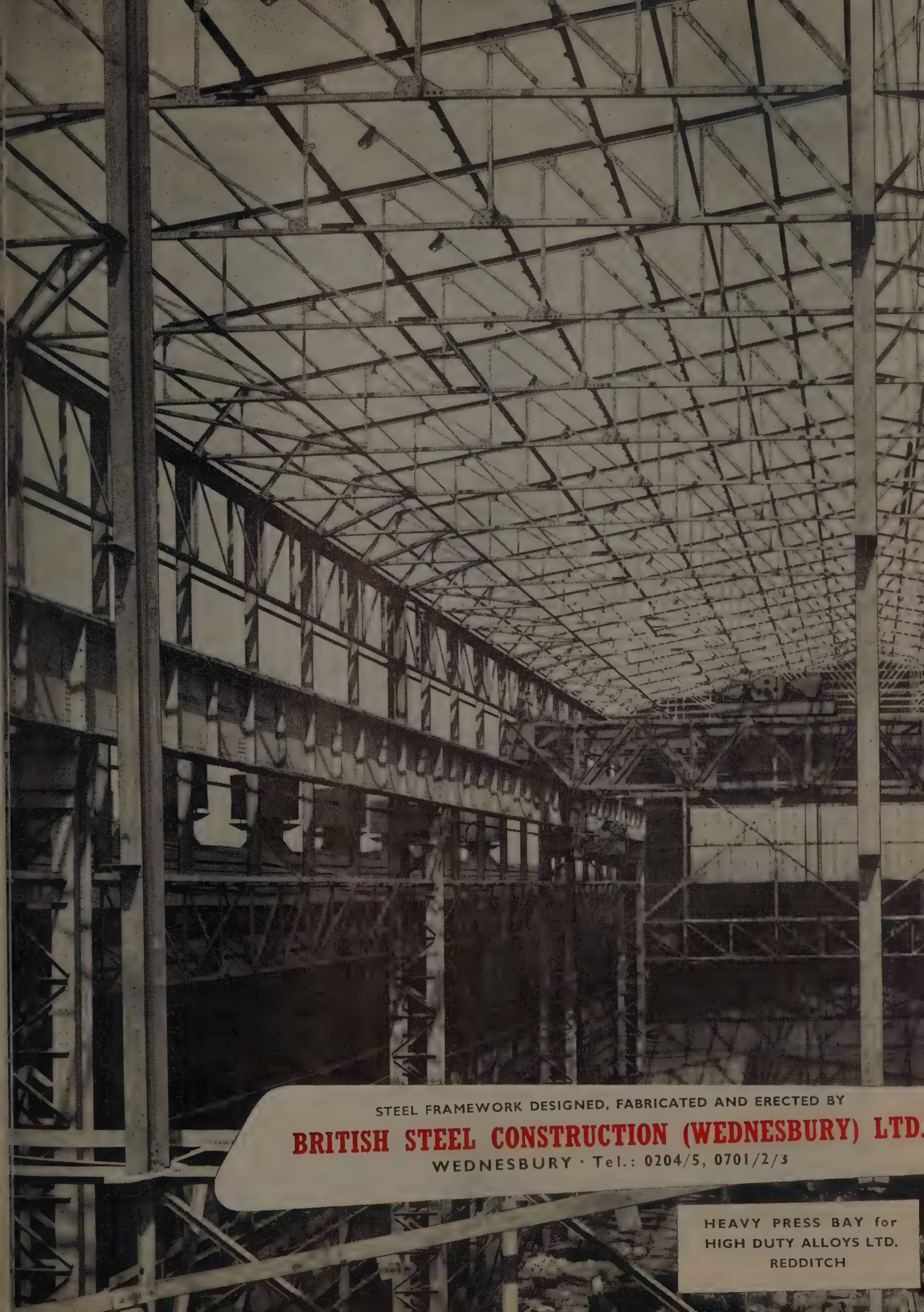

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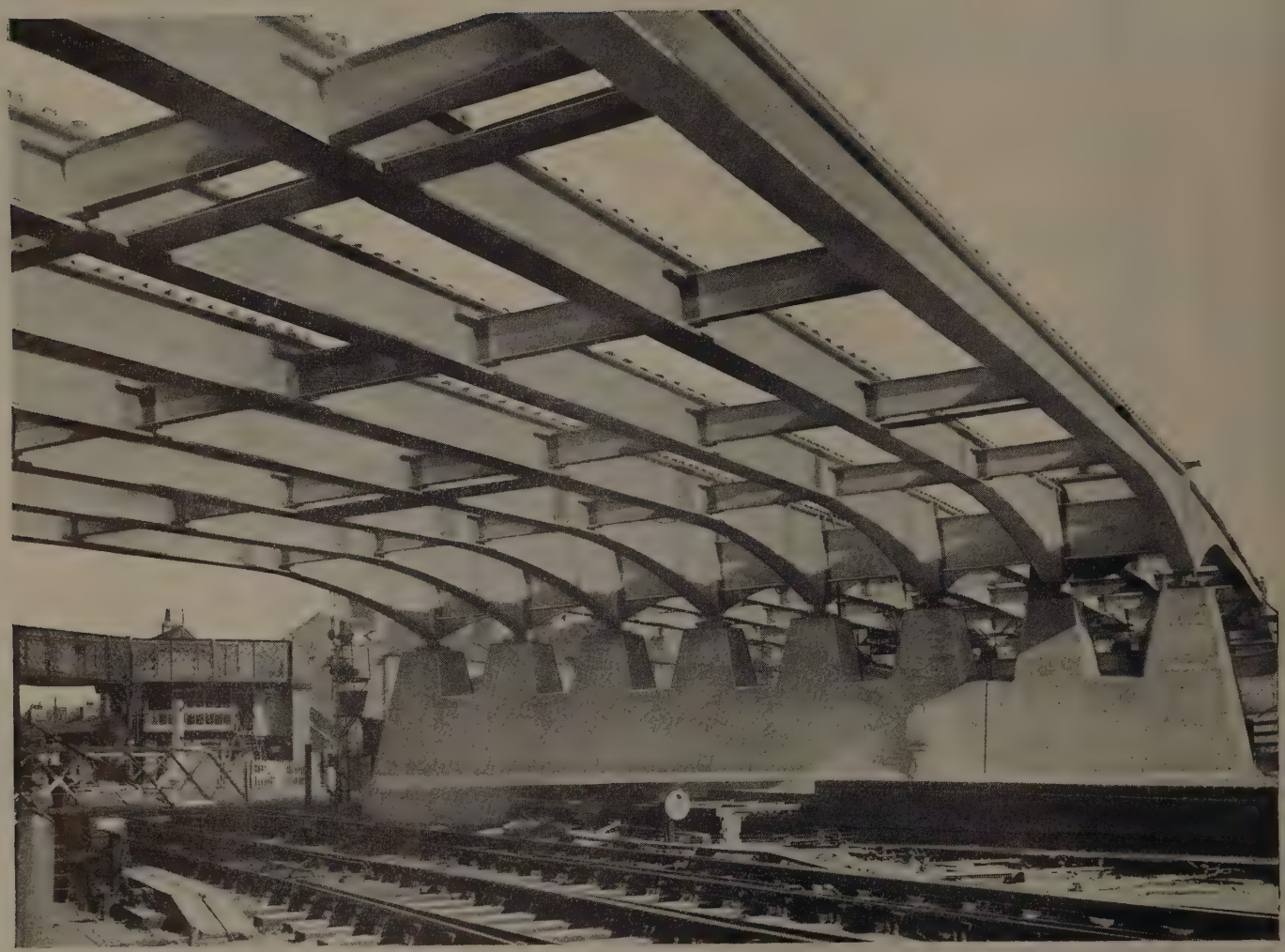

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Auckland Star photograph of the 4 million gallon reservoir under construction

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Design: Pre-Stressed Concrete (N.Z.) Ltd.

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MAJOR PROJECT FOR SUPER-TANKERS ON THE MERSEY

This illustration of the supertanker berths at Tranmere has been reproduced by the courtesy of the Mersey Docks and Harbour Board. Construction of the berths has already been commenced under the direction of their Engineer-in-Chief

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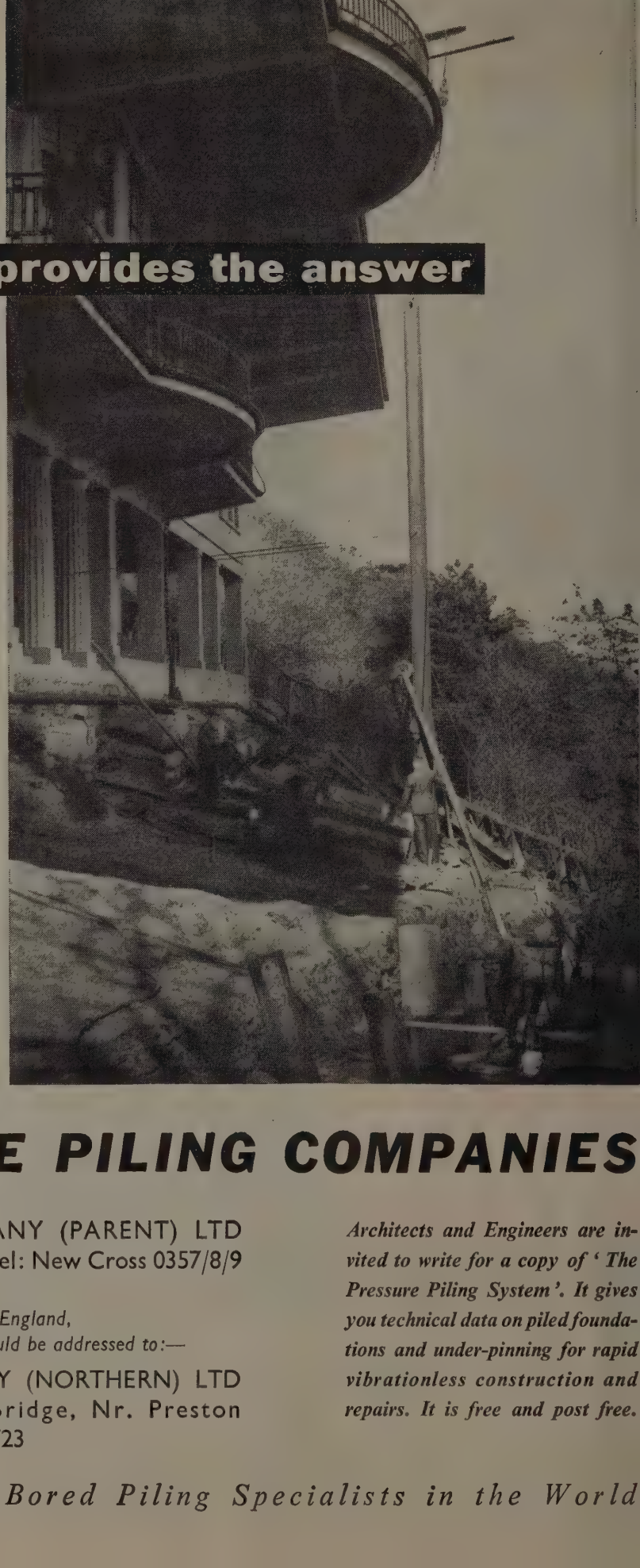

Leas Cliff Hall, owned by the Folkestone Borough Council, is sited on a cliff face. Owing to the cliff showing signs of slipping (note the exposed foundations in the photograph), it was decided to construct a protection apron at ground floor to carry the rainwater away from the existing foundations.

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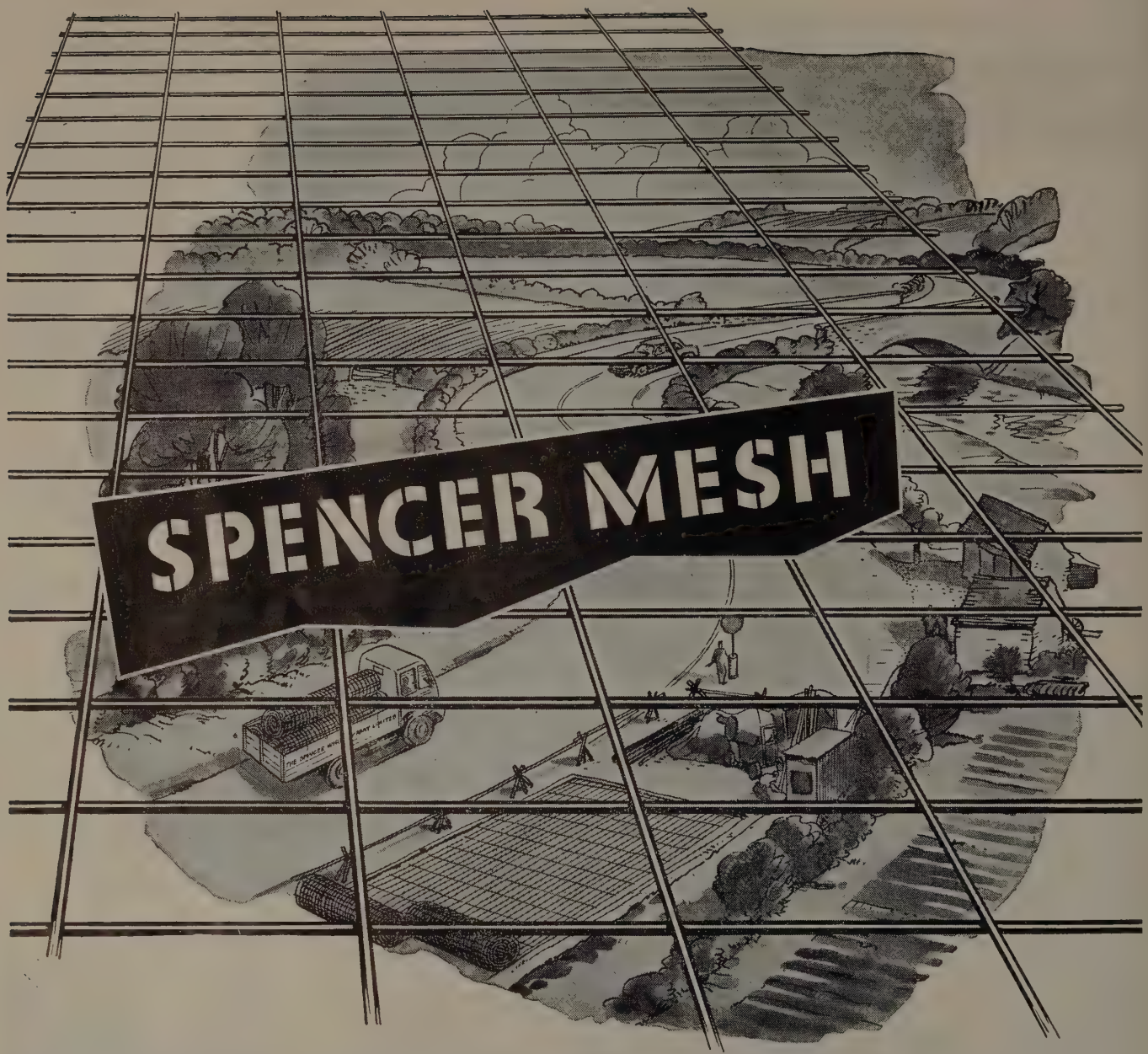

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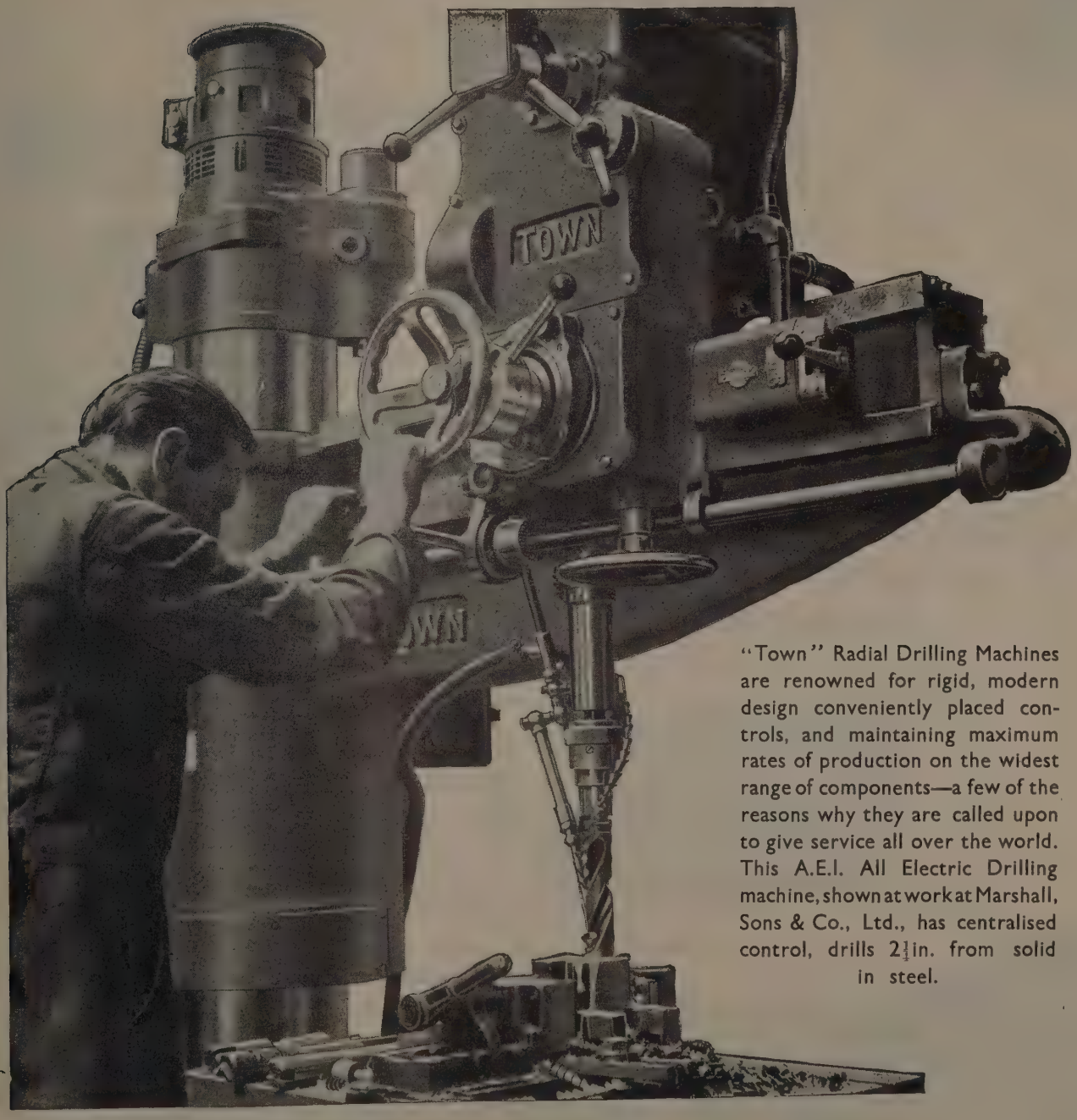

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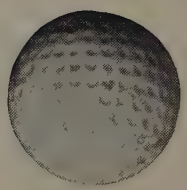

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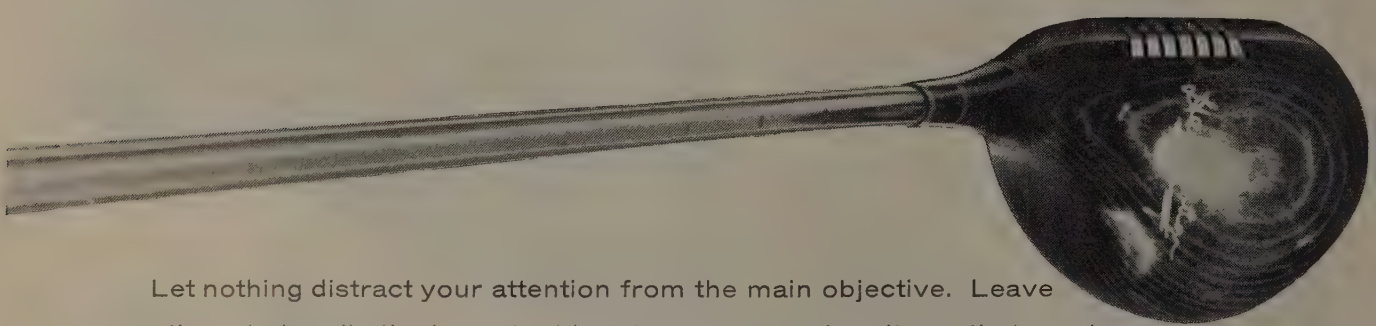

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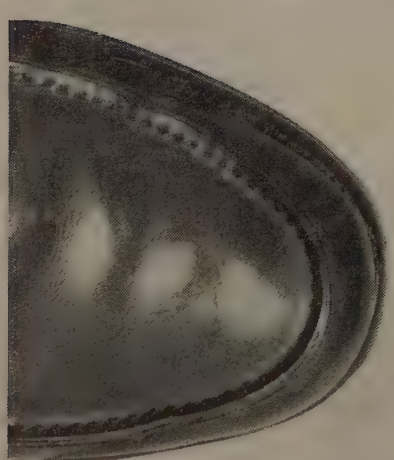

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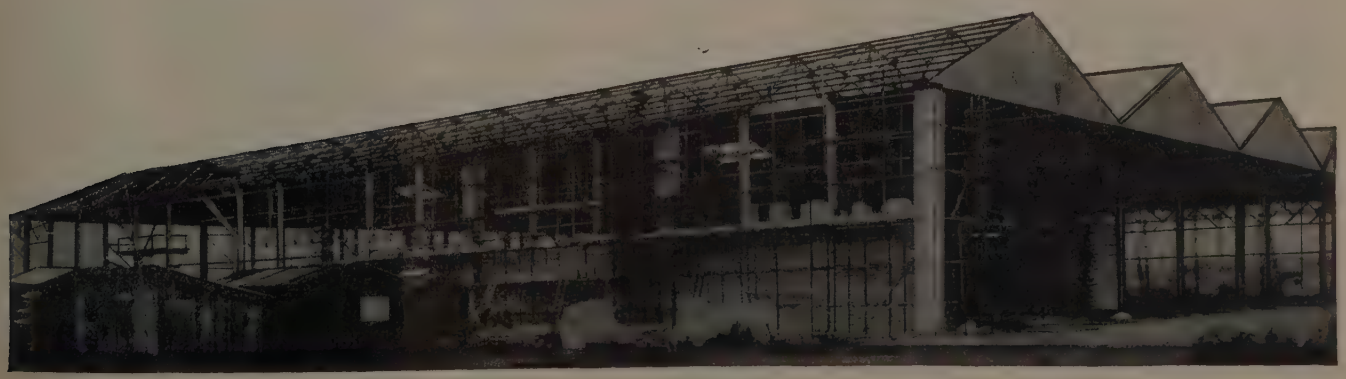

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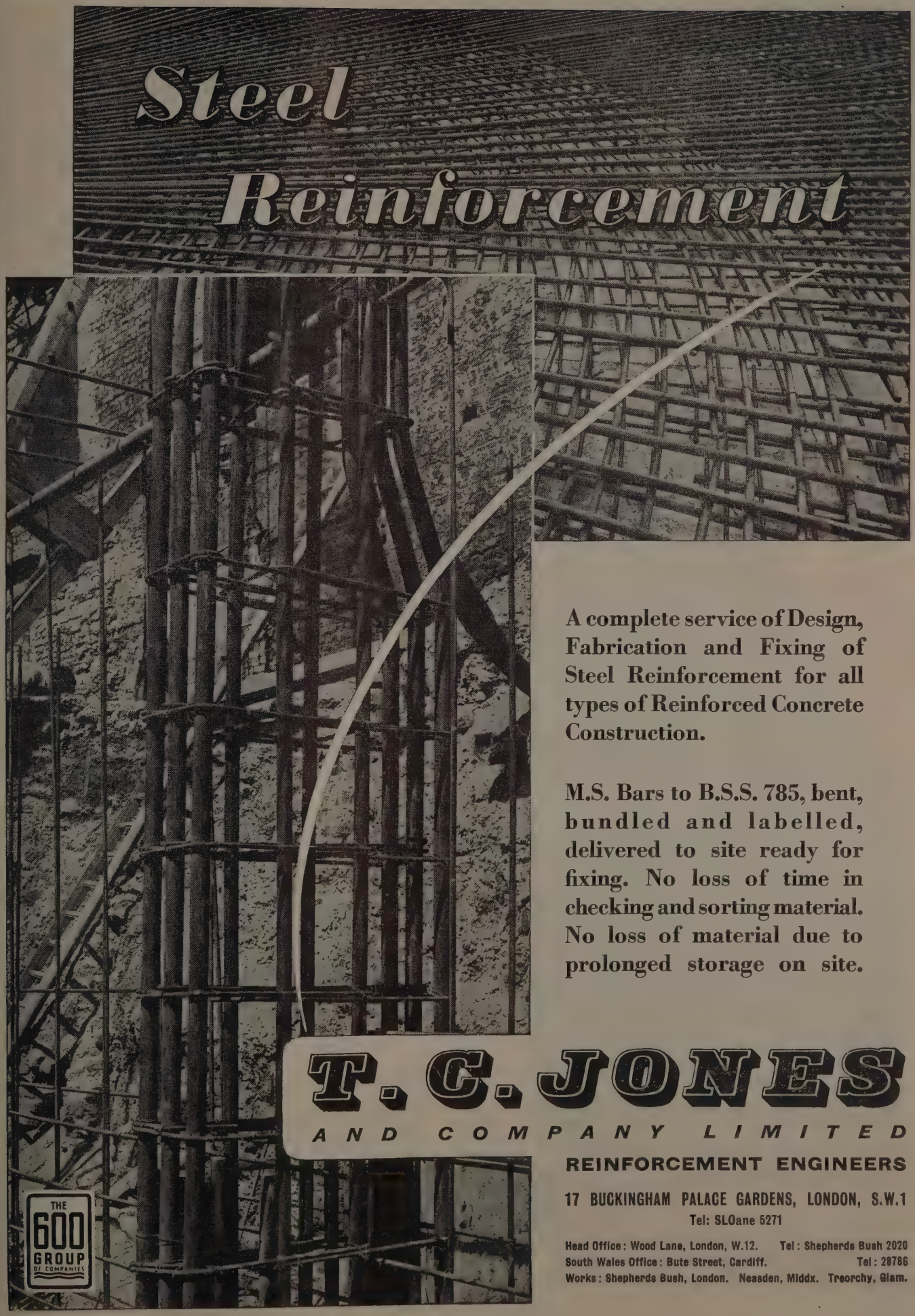

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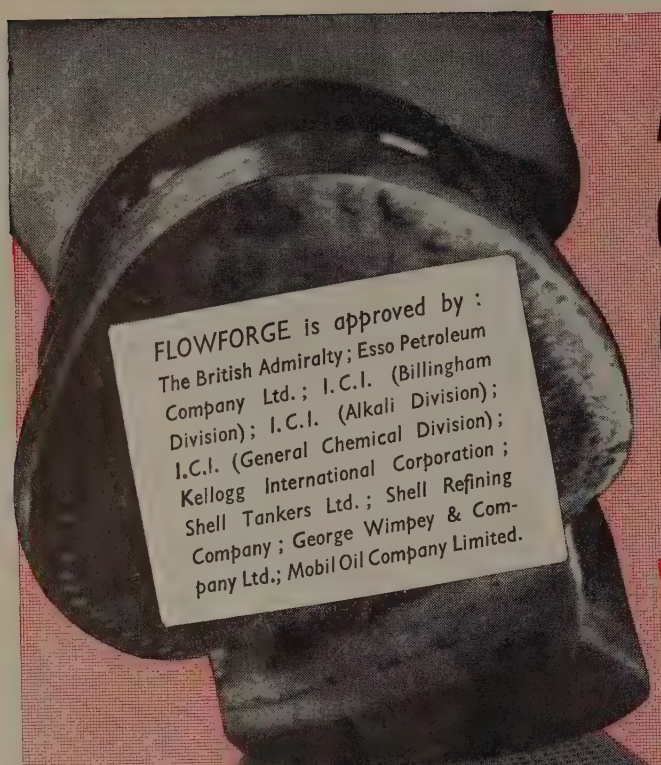
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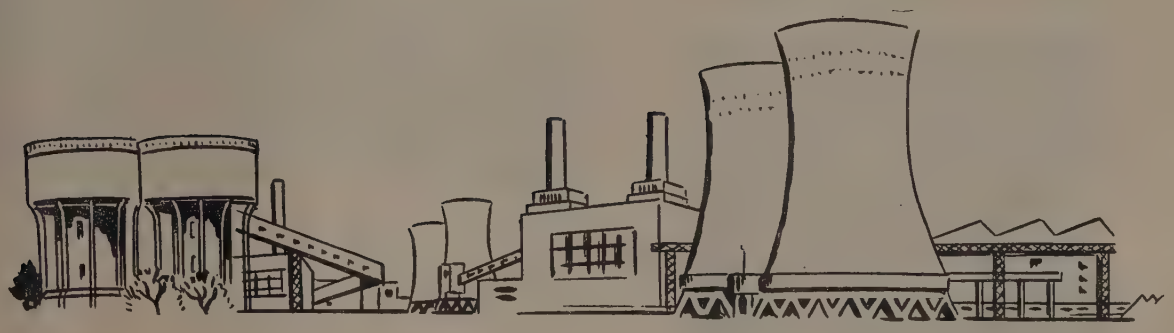

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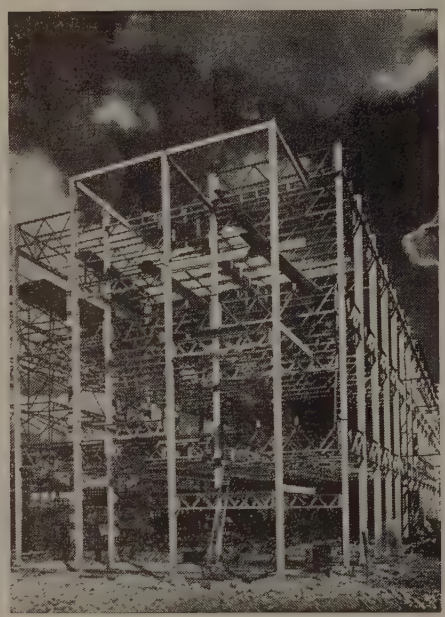

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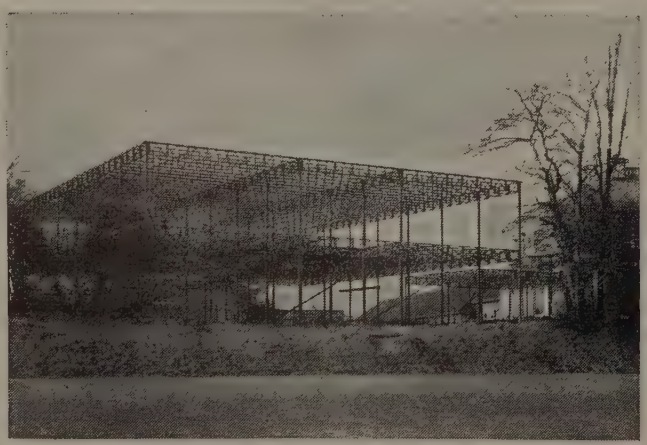

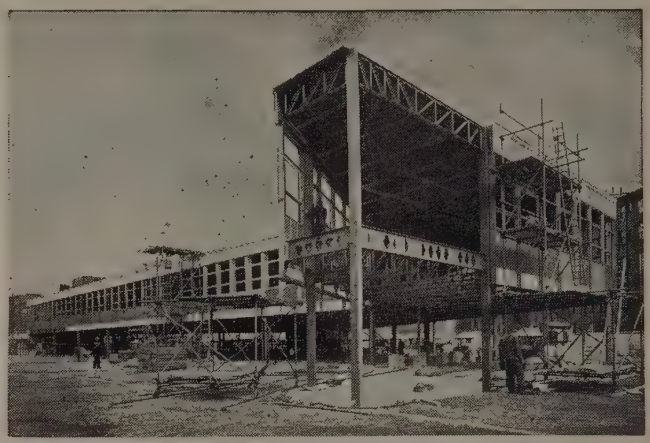

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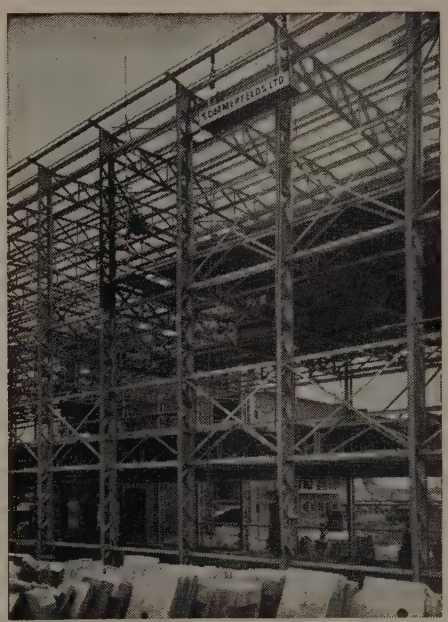

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THE STRUCTURAL ENGINEER

THE JOURNAL OF THE INSTITUTION OF STRUCTURAL ENGINEERS

Founded 1908. *Incorporated by Royal Charter* 1934.

11 UPPER BELGRAVE STREET, LONDON, S.W.1. Sloane 7128/9.

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The Institution of Structural Engineers, whilst being at all times desirous of receiving communications from members and others, does not accept any responsibility for the opinion expressed by any individual Authors or Speakers

Annual Subscription Rate to the Journal £3 3s. post free.

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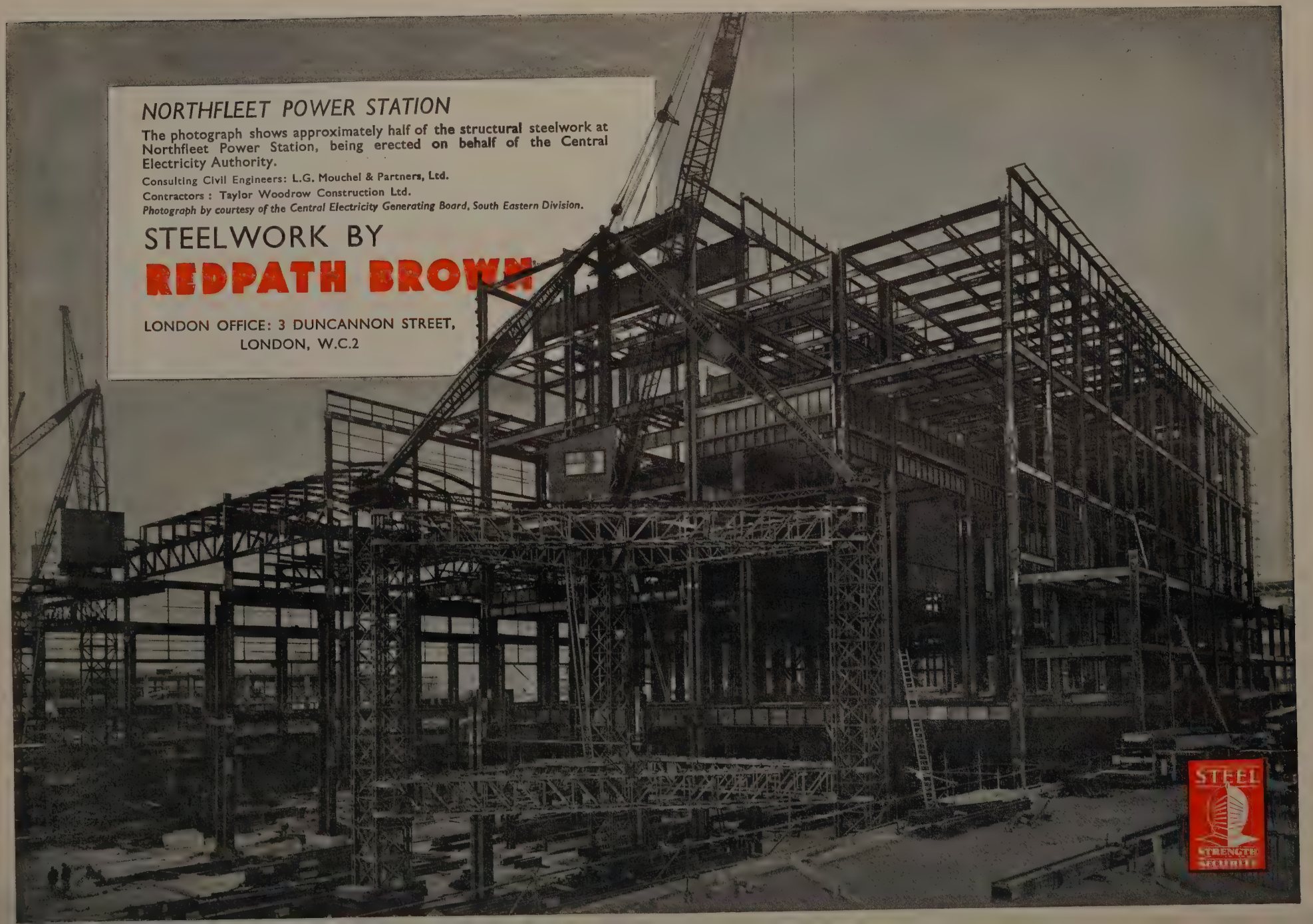
NORTHFLEET POWER STATION
The photograph shows approximately half of the structural steelwork at Northfleet Power Station, being erected on behalf of the Central Electricity Authority.
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The President, 1958-59

Mr. G. S. McDONALD, M.I.Struct.E., M.I.C.E., M.I.Mun.E.

The Institution welcomes as President for the Session 1958-1959 Mr. G. S. McDonald, M.I.Struct.E., M.I.C.E., M.I.Mun.E., who takes office on the 2nd October.

Gordon Stuart McDonald was born at Southampton in 1904 and was educated at King Edward VI School. In 1922 he was articled to Mr. W. W. Gandy, of Eastleigh, Hants., and on completion of his pupilage in 1925 he was appointed Engineering Assistant in the Public Works Department of the City of Birmingham. After twelve years' varied experience in the Department he joined the late Sir Herbert Humphries, C.B.E., formerly City Engineer of Birmingham, as a Consulting Engineer, the practice being styled "Sir Herbert Humphries and McDonald." Since the death of Sir Herbert Humphries in 1938, Mr. McDonald has been Senior Partner in the practice.

Mr. McDonald was elected a Member of the Institution of Structural Engineers in 1939. Since 1945 he has served on a number of Committees, including those dealing with Bye-Laws, Finance, Education and Examinations, Membership, Special Awards and the Aluminium Development Association Research Scholarship Committee. He served on the Literature Committee for ten years, acting for several Sessions as Chairman of the Adjudicating Panel in connection with the award of medals and prizes for papers read and published. He has been Chairman of the Education and Examinations Committee, the Membership Committee and the Finance and General Purposes Committee. In addition, he has represented the Institution on the British Standards Institution Building Divisional Council and the West Midlands Advisory Council for Technical Education.

Since 1947, Mr. McDonald has been a Member of the Council of the Institution of Structural Engineers. He held office as Honorary Librarian in 1951-52 and as Vice-President during the Sessions 1952-58. He has taken a prominent part in the affairs of the Midland Counties Branch, of which he was Vice-Chairman in 1945-46 and Chairman in 1946-47.

He is also a member of the Institution of Civil Engineers and the Institution of Municipal Engineers and a Fellow of the Royal Society of Health.

Mr. McDonald comes to office at a period when the Institution is celebrating what is in effect a double Jubilee—the fiftieth anniversary of its foundation and the twenty-fifth anniversary of its incorporation by Royal Charter. The duties which fall to the President in any year are arduous and during the coming Session the official engagement list will be even more heavily charged. Members who have already met Mr. McDonald know that his energy will prove equal to the calls which will be made upon it. His year of office will bring him many new friends. Members in all grades will wish him well and those who will be serving under him look forward with pleasure to an eventful year in the conviction that under his friendly leadership the Institution will greatly prosper.

Aluminium and Tubular Steel as Applied to the Construction of Some Recent Aircraft Hangars*

By L. E. Ward, M.I.Struct.E.

Synopsis

A brief introduction states the proportion between structure and total building cost and considers the effect of ancillary buildings on hangar design. A description is then given of aluminium as a structural material followed by a detailed account of the design of a 200 ft. span aluminium hangar. Unusual features of three other types of aluminium hangar are then given. Tubular steel as a structural medium is discussed and illustrated by a description of a 150 ft. two way span tubular steel hangar. Conclusions are stated and a bibliography given.

Introduction

An aircraft hangar is used to house aircraft during periods of adjustment repairs and maintenance. Thus so that these functions may be fulfilled efficiently it must be well lighted, heated and ventilated. Services such as compressed air, water and electricity in addition to defuelling and refuelling points will be required either within the hangar or on adjacent aprons. The cost of these latter services is to a small extent dependent on the area of the building. The costs of cladding, floors, artificial lighting, installation and running costs of the heating system vary as the floor area. It can be shown that by eliminating columns and providing a large clear span roof the floor area can be used more efficiently and for the accommodation of a given number of aircraft the area will be a minimum when the spans are a maximum. The cost of the structural frame may vary between 15 per cent and 20 per cent of the entire building costs and the additional cost of large spans is a mere fraction of the whole. Thus with large spans the additional structural cost is more than offset by the overall savings due to reduced area.

The layout of the structure is dependent on the type of work carried out. For routine maintenance aircraft may be housed for a few hours only and it is apparent that a large number of doors will be required. For a major overhaul the occupation period will be considerably longer and ease of access not so important. Likewise a greater area will be required for workshops and stores and this factor has an important bearing on the arrangement of the structure. The early concrete hangars at London Airport consist of two lines of bays with workshop area between. Some of the more recent American hangars are of similar layout but advantage has been taken of this central core to cantilever the hangars either side. Notable examples are those at Philadelphia and Kansas City designed by the American Engineers Amman and Whitney. A similar type of hangar has recently been constructed at London Airport for B.O.A.C.

In the following description aluminium and tubular steel are introduced as structural materials followed by examples of hangars in which these materials have been used.

* *Paper to be read before the Institution of Structural Engineers, at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 23rd October, 1958, at 6 p.m.*

Aluminium as a Structural Material

A brief outline of aluminium as a structural material has been given in an earlier paper by the author[1] and by others. A more recent development, that of the bulb angle, is worthy of mention and there follows an account of this.

The main structural members of the aluminium hangar at London Airport are composed of double battened channels. Apparently economic insomuch as the load bearing metal is efficiently used, a considerable amount of material that is not directly load bearing is required in the form of batten plates. The material so used may amount to 25 per cent of the material it connects. Furthermore, each pair of batten plates requires a mimimum of eight rivets. In general the rivets for the structural joint are in single shear, thus double the number are required than for the ideal case.

Towards the end of 1947 and in 1948 consideration was given to alternative methods of construction and a thorough examination into various methods resulted in the development of the bulb angle. Briefly, this consists of a thin angle section with bulbs introduced at the extremities and at the root. They are generally in the form of unequal angles used in pairs although for smaller loads single equal bulb angles are used. The principal advantages of bulb angles are :—

(a) The stiffening effect of the bulbs enable the leg/thickness ratio to be increased from 12 without bulbs to 20 with bulbs whilst still permitting the proof stress to be achieved without local instability.

(b) Consequent on the spreading of the metal a higher radius of gyration is obtained thus permitting a higher permissible compressive stress.

(c) The introduction of bulbs at the extremities increases the speed of extrusion and thus reduces the basic cost.

(d) The rivets in main connections are in double shear and fewer rivets are required in the packings. The size of legs being increased permits easier riveting.

A 40 ft. span bulb angle truss was fabricated and subjected to loading tests which proved entirely satisfactory. Various proportions of sections were then investigated and ultimately standardised. A very similar bulb angle was developed by the Aluminium Development Association and published as an A.D.A. Standard in 1953. Details of these bulb angles and a typical angle to B.S. 1161 are shown in Fig. 1. It is of interest to note that all the sections in the range have the same proportions and once the mechanical properties of one size are calculated the remainder can be obtained by ratio.

The bulb angle section was first advocated by Temple[2] in 1947, although the proportions differ from the S.M.D. and A.D.A. bulb angles.

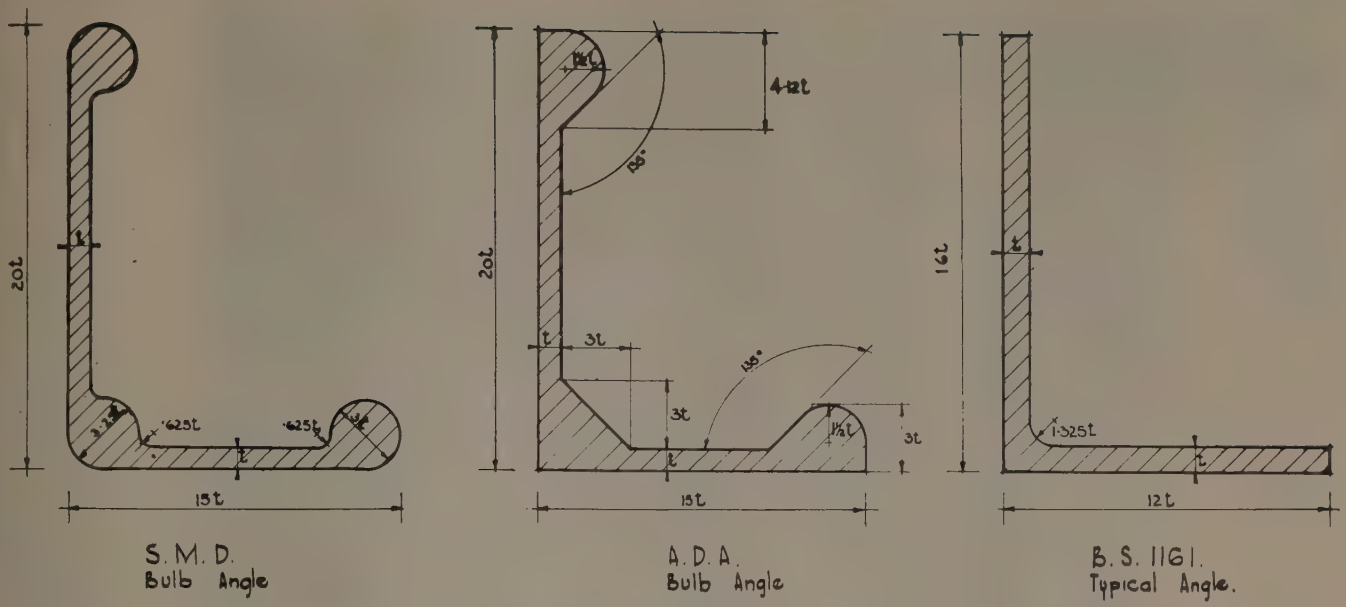

Fig. 1.—Comparative Details of Angles

Working Stresses—Aluminium

A considerable number of aluminium structures had been erected before the Report on the "Structural Use of Aluminium Alloys in Buildings" was published by this Institution in 1950. Furthermore, a considerable number of tests had been carried out on components and full scale structures and consequently a code of practice had in effect been formed. In the case of the London Airport hangars the Air Ministry had specified that the factors of safety were to be at least equivalent to those in a steel structure and consequently compression members were designed on a factor of 2.0 on the guaranteed proof stress and tension members with a factor of safety of 3.0 on the minimum ultimate tensile stress. In the case of rivets the shear value was taken as 3.33 ton/sq. in. and the bearing stress as 10.8 ton/sq. in. This latter figure, though appearing somewhat high, was decided upon after a number of tests had been carried out.

Following the publication of the report these figures were still adhered to excepting that the tensile stress was increased in line with the report. With the exception of the Ministry of Works hangar at Thurleigh the aluminium structures were designed in accordance with the following :—

Compression Members. Factor of 2.0 on Minimum Proof Stress of 15 ton/sq. in.

Tension Members. 6.5 ton/sq. in.

Shear Stress in Aluminium Rivets (A.W.5.) 3.33 ton/sq. in.

Bearing Stress in Aluminium Rivets 10.8 ton/sq. in.

The customary 25 per cent increase of stress was allowed for wind load conditions.

The extrusion material used throughout was designated AW10 later known as HE10WP. In the case of the Ministry of Works hangar the manganese content was increased to the maximum allowed within the Specification and this resulted in a higher proof and ultimate stress and a great modulus of elasticity. The HE10 specification was later rewritten and further

Fig. 2.—Comet Flight Shed (Monro & Partners, Architects and Engineers)

Fig. 3.—Comet Flight Shed—Transverse View Showing Roof Construction

subdivided. Thus the well known HE10 was retained, an alternative and popular variety was known as HE20, the high manganese content alloy became HE30 and the minimum proof and ultimate for this alloy was raised to 16 and 19 ton/sq. in. respectively. Experience has shown that these values are conservative and stresses as high as 1 ton/sq. in. above the minimum are regularly attained and indeed guaranteed by certain suppliers.

Comet Flight Shed

In 1951 the De Havilland Aircraft Co. Ltd., through its architects Messrs. James M. Monro & Son* were considering the construction of a new Flight Shed for Comet aircraft.

Enquiries were made on the basis of design and tender from both steelwork and reinforced concrete contractors and finally from aluminium contractors. Within a week of discussions with the Architects a preliminary design and estimate was produced. Minor modifications were introduced by the architects to improve the natural lighting and appearance. Following subsequent design modifications, the quotation was accepted and within one year of this date the shed was occupied.

The shed is 200 ft. clear span × 330 ft. long × 45 ft. clear heights with doors at each end. This area was designed to house seven Comet 1 aircraft and provided ample insurance against aircraft of larger span or heights. In order to obtain adequate and constant lighting, North light glazing was to be incorporated. Fig. 2 shows a general view of the completed shed and Fig. 3 shows the roof framing.

Portal frames spanning 200 ft. are spaced at 30 ft. centres the depth of the top girder being 10 ft. and the width of the legs 8 ft. 6 in. Trusses are spaced at 10 ft. centres the rafters running from the top beam of one portal frame to the bottom of the next. With

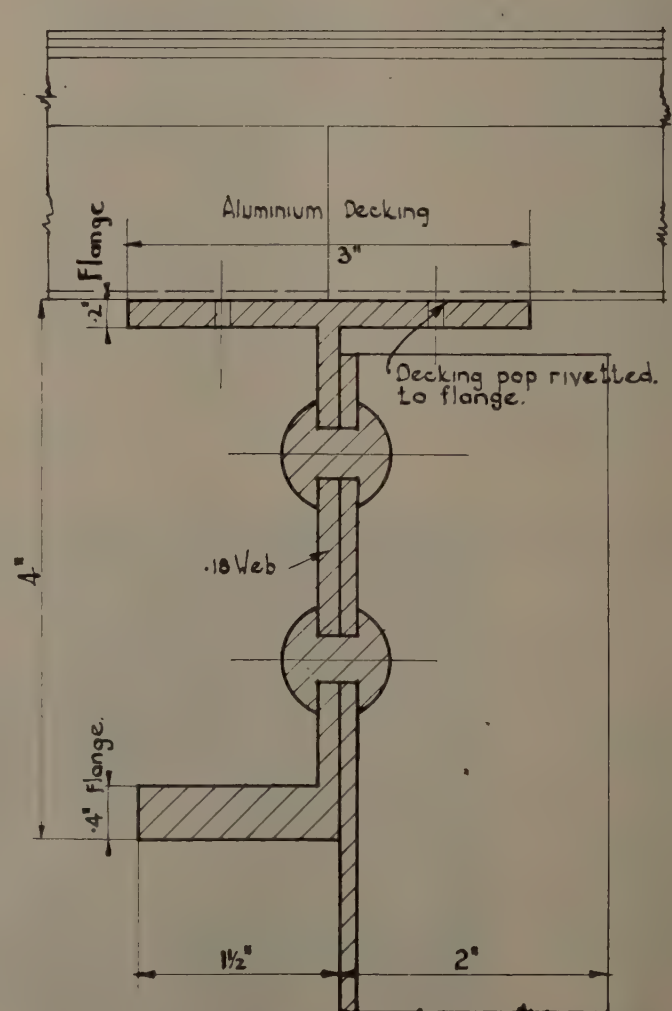

Fig. 4.—Comet Flight Shed—Section Through Truss Rafter

* Now Monro and Partners, Architects and Engineers.

the need for economy constantly in mind it was first decided to eliminate purlins and span the roof cladding from truss to truss. The maximum available span of aluminium decking was 10 ft. and this dimension determined the centres of the roof trusses. It was also decided to use the decking to provide lateral stability to the truss rafters and for this reason it was fixed with pop rivets through site drilled holes, two light angle ties being used as truss spacers.

Pursuing the design of the truss rafter, lateral instability having been eliminated, the logical section to use for economy would be an I section. However, to obtain ideal conditions the next step would be to eliminate gussets and in the main this was achieved by removing one side of the bottom flange and thickening the other as a compensatory measure. This permitted the internal members of the truss to be riveted direct to the web as shown in Fig. 4. In order to provide the minimum obstruction to daylight the lower glazing member was stooled off the rafter and for economy the top glazing purlin was eliminated, cleats being riveted to the main portal frame.

Attention was then given to the design of the main portal frames and in view of the fixed geometrical proportions of the shed a two pin portal frame provided the most economical solution. Fixed bases would not have resulted in economy and furthermore would have entailed higher foundation costs.

By the use of 2 pinned portals a bending moment is induced under U.D.L. at the knee equal to .074WL and therefore the mid-span moment is reduced to .125WL — .074WL = .051WL. the distance between points of contraflexure being approximately .71L. Thus the bending moment over the majority of the top girder is substantially reduced and, although the maximum moment occurs at the knee, its effect is somewhat local.

The vertical loads on the shed were computed as follows :—

Dead load including decking, glazing and structure.—10.2 lb/sq. ft. on plan.

Superimposed load (B.S.449 : 1948) and allowance for future runways.—12.2 lb/sq. ft. on plan.

Uplift due to wind (velocity = 65 m.p.h.).—14 lb/sq. ft. on plan.

The nett downward load was therefore 22.4 lb/sq. ft.

The nett upward load was therefore 3.8 lb/sq. ft. a reversal of 17 per cent.

This reversal of load is a feature of the design of large aircraft hangars and this combined with the effect of side wind loads results in the majority of members being designed for compression. The depth of the main girder was fixed at 10 ft. which admirably suited the design of the roof truss and of course both the top and bottom boom were stabilized at 10 ft. intervals by the trusses. The legs are 8 ft. 6 in. deep and on the one side are stabilized by sheeting rails and ties and on the other by an outrigger.

Aluminium struts are usually designed in accordance with the Perry-Robertson formula and this is specified in the Institution of Structural Engineers "Report on the Structural Use of Aluminium Alloys in Building, 1950."[3] This curve is shown plotted for AW10E, a material with a guaranteed proof stress of 15 ton/sq. in. in Fig. 5.

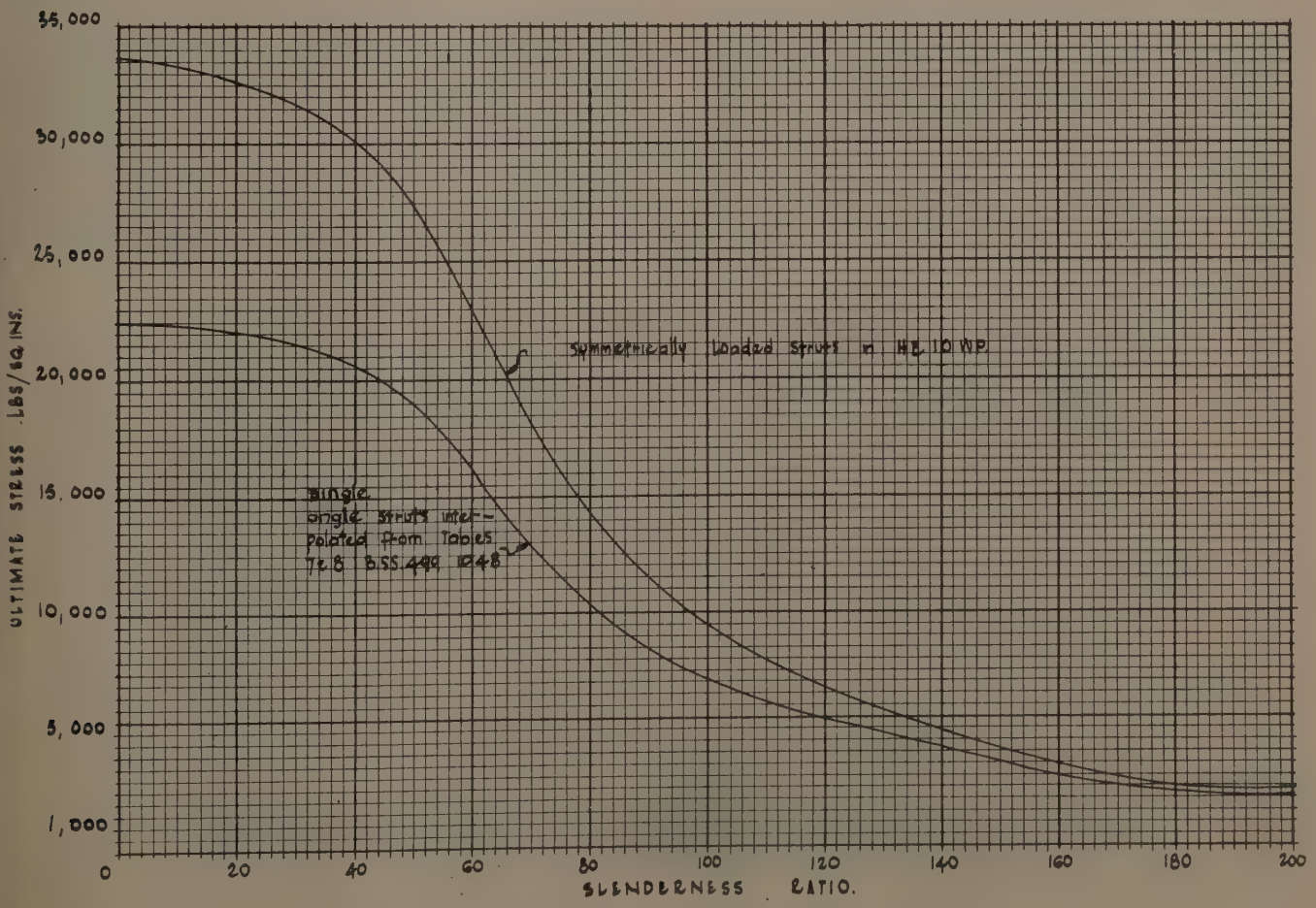

Fig. 5.—Perry Robertson Curve for HE 10 WP

It will be seen that there is a rapid falling off of allowable stress when the slenderness ratio exceeds 40 and for this reason it is economically desirable to arrange a structure so that this limit is rarely exceeded.

The portal frame was designed by strain energy methods against an assumed horizontal thrust H. This method is a simplification of Castigliano's theorem. The forces having been determined, the individual sections were designed. The maximum size of bulb angle obtainable from the extrusion manufacturers was 7 in. × 5¼ in. and where this section was insufficient strengthening was carried out by plating. Due to difficulty in obtaining plates of the required size and length the actual plate 9 in. × .29 in. thick was extruded. A further limitation on extrusions was that sections greater than 36 ft. long could not be obtained fully heat treated and therefore each section was limited to a maximum length of 30 ft. between joints, all of which occurred at node points. Although this added to the number of joints, and hence the cost, there was some saving in that each section was designed for the worst condition of loading in one bay in each of three, which is less severe than one bay in each of five. The slenderness ratio of the boom compression members is in the main between 40 and 45 rising to 60 in isolated instances.

Single angle struts are only economical when the loads carried are comparatively small and for want of better information a further strut curve was produced for single angles double bolted or double riveted. This curve, which is also reproduced in Fig. 5, was obtained by interpolation between Tables 7 and 8 of BS 449 and the Perry Curve for axial loading.

The feet of the portal frames were ideally pinned by a steel bolt through welded steel brackets, the centres of the pins being 217 ft.

The sheeting rails which span 30 ft. between frames were designed in a similar manner to those at London Airport. The main members were 4¾ in. × 2¼ in. × .2 in. thick Channels at 6 ft. centres and were supported both vertically and horizontally at points 7 ft. 6 in. from the portal frame. Vertical supports in the form of ties were inclined in the upper bay back to the portal frame. The horizontal supports were in the form of raking angles to the inner boom of the portal frame. Longitudinal ties along the inside leg of the portal frame transmit any out of balance loads to the end bays which are fully braced. Thus this system is economical in sheeting rail design and performs the dual function of stabilizing the inner boom of the portal frame. The sheeting is corrugated aluminium with a ¼ in. incombustible asbestos based lining board to which is affixed aluminium foil insulation the whole being secured by aluminium tees to the inside leg of the sheeting rail. This cladding is economical in cost, weighs only 4 lb. a square foot and has a thermal insulation 'U' value of .28 B.T.U.'s /sq. ft./hr./°F.

The one side of the shed abuts an annexe building and in order to provide clear openings for access to this the portal frame leg is stabilized by out-riggers. These are constructed to provide clear openings of 30 ft. and 22 ft. in alternative bays.

Wind girders are designed to take a pressure of 1.0 p. from the doors together with accumulated pressures from each roof slope. The girders are 60 ft. deep and the bottom booms of the relevant trusses and portal frames are strengthened to cater for these additional loads. Side bracing in the end bays transmits the load to the foundations through steel flats. These flats were secured to the foundation in the early stages of construction and after plumbing of the main frame the steel and aluminium were connected by turned bolts in site drilled holes. The foundations were isolated blocks sufficiently large to anchor the structure against uplift and as an additional safeguard the opposite bases were connected across the width of the shed by prestressed tie beams using the Fressinet method. The end foundations were subjected to additional wind loads and the two end blocks were connected together with a beam in order that the horizontal loads should be equally shared.

With the relatively low modulus of elasticity of aluminium the calculated deflections were appreciable, the downward deflection due to snow load in the end portals being 5 in. and the upward movement due to wind 7 in., whilst the settlement due to dead load was calculated to be 6.8 in. In the case of the intermediate frames these dimensions were somewhat exceeded. In order to allow for this movement at the door head the door girders were formed with 15 in. channels and

Fig. 6.—Comet Flight Shed—Erection of Main Frames

so positioned that a total movement of 12 in. could be accommodated. All the frames were manufactured with a camber of 15½ in. ensuring a permanent camber and assisting with roof drainage. The shed is equipped with power operated folding doors at each end, those at the north providing a generous amount of glazed area.

The roof is drained by combined gutters and walkways around the perimeter of the shed. These are constructed as vertical and horizontal girders, the outer vertical being of plate construction and the inner latticed. The gutters and handrails were delivered to site in 30 ft. lengths, hoisted and bolted in position with 8 bolts. Waterproofing of the whole of the roof structure and gutter is by means of mineral surfaced felt and baffles are provided at the bottom of the patent glazing to prevent the rainwater washing over the gutters. In view of the site problems associated with surface water drainage the whole of the drainage system is fully sealed and the roof thus acts as a reservoir in case of need. For this reason downpipes are somewhat smaller than usual, 6 in.-4½ in. dia. being provided along each side. In order to guard against blockage weirs are introduced in the gutter walls at the ends of the building. The flashings between the shed and the annexe building are constructed to take account of differential movements.

Erection was carried out on the lines associated with traditional steelwork save that considerably less equipment was required. The legs of the portal frames were erected first and then the entire transverse girder weighing 4½ tons was lifted with two 5 ton cranes, the tackle being spread by lifting beams. Roof trusses bracings etc., were lifted by lightweight tackle fixed to the portal frames. The rate of erection was approximately one bay per week.

The Comet Flight Shed has aroused world wide interest in that it is the largest aluminium hangar constructed. By ingenuity in design, attention to detail and the utmost co-operation between the Architect and Contractor, a very fine result was achieved. It is particularly fortunate that the appearance is enhanced by the addition of an annexe building forming air traffic control, pilots' accommodation, fire station and stores.

Wing Hangars

By the time the Comet Flight Shed had reached the final design stage a request had been received from the U.S.A.F. for a scheme to provide a single hangar to accommodate the nose, wings and engines for maintenance of a wide variety of military aircraft. A further complication was the existence of a small amount of paving and two cantilever wing shelters on the site. A scheme was therefore prepared for a wing hangar of 165 ft. clear span built around and isolated from the existing structure. The completed hangar is shown in Fig. 7.

The scheme consists of a 165 ft. span portal frame linked with 30 ft. span trusses to a 90 ft. span girder spanning along the front of the existing wing shelters. At the rear the nose is accommodated in a structure extending from the existing one. For economy the cladding is asbestos sheeting, uninsulated, and supported on purlins spanning between trusses. The top boom of the truss is Channel construction, this having been selected to take local bending and for ease of connection of the truss internal members. A wind girder in the place of the bottom tie transmits the wind loads to the end bracing and foundation.

Fig. 7.—Wing Hangar for U.S.A.F.

The hangar was designed to accommodate aluminium bottom rolling sliding doors which were installed later. The deflection of the structure is appreciable and the differential movement between hangar and doors is accommodated in a novel manner. As each leaf requires approximately 100 ft. of top guide it was necessary, for economic reasons, to reduce the cost of the guide to the minimum. This was done by using a specially extruded lipped channel, the web of which was horizontal. The dimensions were adequate to accommodate a 6 in. dia. horizontal ferrobestos roller on each side of which were small vertical guide rollers. The assembly is connected to the door frame by means of a shaft and sleeve bearings which permit vertical sliding action and simultaneously transmit any horizontal wind loads to the structure. Thus the rollers ride with the structure and take up the vertical movement. The doors are equipped with removable panels in order that fuselage of aircraft may project through at selected positions. These panels are interchangeable and the appropriate type, complete with canvas flashings, can be readily fitted when they provide an adequate seal.

A feature of this particular contract was the speed in design and construction. The design and estimate was completed in four days, which included a weekend, erection was commenced six weeks later and handed over completely sheeted with the floor laid in thirteen weeks from commencement of design. A further seven identical hangars were erected on this and other airfields.

Transportable Admiralty Hangar

Another interesting example of aluminium hangar construction is to be seen in the transportable hangars designed and manufactured for the Admiralty. Although the size is modest the design features several unusual details.

The Admiralty specified a clear span of 60 ft. together with a clear height of 20 ft. with a proviso that any additional space provided at the sides of the hangar would be considered in reviewing the tenders. The method of erection was to be as simple as possible in that the minimum amount of equipment was to be used and it was to be carried out by labour unskilled in this field. For the same reason the number of different parts was to be a minimum. A further limitation imposed was that each component was to be capable

of being manhandled, was to occupy the minimum space when stowed and save in exceptional circumstances no piece was to exceed 13 ft. in length. The loading conditions were onerous in that a snow load of 15 lbs/sq. ft. was specified together with wind loads due to a 110 knot wind. Finally, as they were to be used as temporary hangars, no permanent foundations were permitted.

Two methods of design were first considered, a 60 ft. span latticed portal frame and a 90 ft. span radial 3 pin arch. The latter was selected because foundation settlement would not lead to serious overstressing. It also permitted the number of different components to be reduced to a minimum and even if the cost was higher than the alternative, account would be taken of the extra usable space at the sides. An internal view of the completed hangar is shown in Fig. 8. The main frames each consist of eight identical collapsible space frames each about 12 ft. 9 in. long with half frames located at the centre and the base. Purlins are again eliminated, $1\frac{3}{4}$ in. deep aluminium trough section sheeting being used to span between the frames. The maximum span of the sheeting is 10 ft. and this dimension fixed the centres of the arches at 11 ft. The $5\frac{1}{8}$ in. distance between the centres of corrugation became the module for the span of the arch, it being necessary to fix the top chord length of each segment as a multiple of this dimension. The space frames were used in order to eliminate a large number of small members but a limited number of ties stabilized the inner boom. Site connections frame to frame are made with a single bolt at each corner through extruded male and forged female connectors shop riveted to the frames. The frames are constructed with single bulb angles laced together in the side and bottom planes. In the top plane the gutter is bolted to the boom angles and performs the dual function of a batten plate. In order to reduce the bulk for stowage the diagonal bracings perpendicular to the axis of the frames are provided with a keyhole arrangement so that the bolts can be slackened, thus permitting the frames to be folded when they assume the form of a parallelogram. Once again gusset plates were eliminated. As an additional weathering precaution baffle plates along the ends of the sheeting prevent the entry of driven rain or snow. Natural lighting is provided by corrugated perspex ridge glazing and in panels in the gable end.

As the hangars were designed to be used on uneven ground it was impractical to have bottom rolling doors and they were therefore top hung. The top track was specially extruded and within the confines of the arch it is an integral part of the wind girder. The remainder of the track spans between the arch frame and an "A" frame at its extremity. The bottom guide can be used in the erection of the hangar and when finally incorporated as the guide is levelled as best the site permits. The guide is a robust extruded section in order to withstand the passage over of rubber tyred vehicles. The bottom track roller is constructed in a similar manner to the top assembly rollers of the doors on the wing hangar but in this case there is an additional vertical flashing fixed to the shaft so as to weather the area between the bottom of the door and the varying position of the guide.

In order to cater for the high uplift loads on the bases, which could amount to 6 tons, the main foundation plates are held down by means of hold fasts and steel pickets as shown in Fig. 9. Turnbuckles are used to take account of variations in the picket positions after driving. This type of foundation, although not new, was subjected to several tests on different types of ground and in each case proved capable of withstanding the imposed loads. In the case of a foundation in loose sand, 'kentledge' would be provided by sandbags. Once the main baseplate is fixed a base casting is bolted on through oversize holes and after being accurately positioned the bolts are tightened, horizontal forces being transmitted by friction. As an additional safeguard, the bases of each arch are connected together by a cable, fixed by means of lugs on the casting.

The method of erection was to assemble the frames in pairs and sheeted with the apex pin about eight feet above ground level. On the one side the base pin is fixed to the foundation and on the other a wheel is incorporated. A horizontal force is then applied to this base wheel which moves inwards on a temporary runway in the form of the door guide and causes the

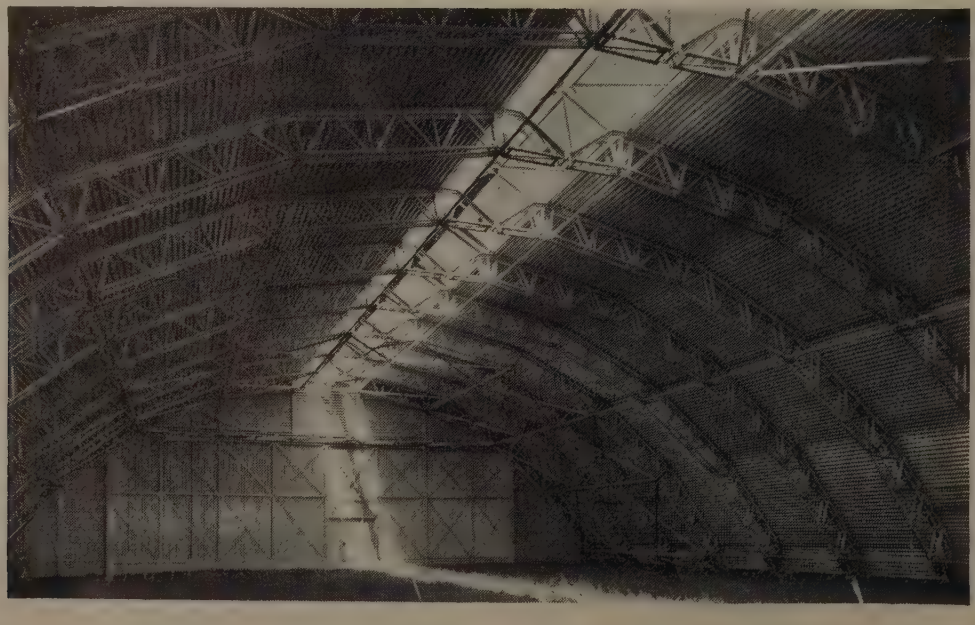

Fig. 8.—Admiralty Transportable Hangar—Internal View of Completed Hangar

Fig. 9.—Admiralty Transportable Hangar —View of Arch Base

apex to rise. When the correct position is reached, the wheel is automatically guided into the base casting and is then secured by a pin through its shaft. Alternate bays are thus erected and the remaining bays are sheeted in situ, the horizontal corrugations proving useful when ascending the roof. For winds in excess of 90 knots, additional safeguards are required in the form of guy ropes which take account of excess wind loads.

Hangar for the Ministry of Works at N.A.E. Thurleigh

Aluminium had for some time been accepted as a structural material and when, in 1954, the Ministry of Works were considering the erection of a Maintenance Hangar at Thurleigh, tenders were invited from specialists in aluminium construction in addition to those in steel and concrete.

The functional specification was for a hangar 200 ft. clear span and 50 ft. clear height and a length of 235 ft. Mechanically operated doors were required at each end. The loading was to be in accordance with B.S. Code of Practice CP3 (Chapter V—Loading). The wind loading requirements of this specification are more onerous than BS 449 insomuch that the uplift is 1.5 p. Account was also to be taken of wind drag. The hangar was to be designed in compliance with the Report on the Structural Use of Aluminium Alloys in Building. In order to reduce both the capital and operating cost of the heating equipment the cube was to be a minimum and the roof, walls and doors were to be thermally insulated. Natural lighting was to be provided in the form of patent glazing in the roof and walls.

After considering the specification it was decided to keep the construction as simple as possible and as the loading requirements did not permit any reduction for a pitch roof a flat roof was decided upon. The glazing, which of course cannot be laid flat, is in the form of four deck lights running across the building. An internal view of the hangar is shown in Fig. 10.

Fig. 10.—Ministry of Works Hangar at N.A.E., Thurleigh

The general method of design followed on similar lines to those already outlined in this paper. Stayed purlins span the 35 ft. between portal frames and ties at the lower chord cater for out of balance loads. In order to eliminate the problem of longitudinal expansion the wind girder and side bracing was arranged in the centre bay. Thus thermal movements either outward or inward can freely take place and the need for expansion joints is eliminated. The wind loads from the doors and gable and due to drag on the roof are transmitted to the centre wind girder through the purlins. Another obvious advantage of this arrangement is that only one system of bracing is required.

When designing the purlins and their ancillary bracing and ties it was found that due to their appreciable length they were either both uneconomic when designed as bulb angles or their slenderness ratio exceeded that permitted by the "Report." The ideal section to reduce the slenderness ratio and to achieve economy of material would be of a tube either round or square. Both these alternatives suffered from the difficulty of making connections and both were costly extrusions. The practical solution lay in using deep, top hat sections which, apart from their resistance to torsion, possessed similar properties to tubes. The connection problem was simplified by arranging the dimension between webs so that the sections interlocked and were then able to be bolted together without the need for gussets. Initial problems of extruding these sections were overcome by the suppliers.

The main frames were designed in a similar manner to the Comet Flight Shed but the additional loads and increased spacing required the use of $8\frac{1}{2}$ in. $\times$ $6\frac{1}{8}$ in. bulb angles. The centres of purlins being fixed at a maximum of 10 ft. to suit the span of the decking, the increase in the size of the bulb angles resulted in a reduction in the value of the slenderness ratio. The Ministry of Works, with customary thoroughness, required tests to be carried out in order to ensure that section would not fail in torsion at a stress lower than that permitted by the Perry Formulae. Specimens, manufactured from two 5 in. $\times$ $3\frac{1}{2}$ in. bulb angles riveted together through washer packings, were tested in compression. The results of the tests appeared perfectly satisfactory until it was found that the actual proof stress was considerably higher than the guaranteed minimum. It was necessary to embody material more nearly approaching the guaranteed proof stress. Although at first sight this appeared a simple matter, it proved difficult but was eventually achieved by reducing both the temperature and period of heat treatment. Further specimens were then tested and the results proved acceptable.

The increased loads and greater centres of frames also made it preferable to use gussets of $\frac{5}{8}$ in. or more in thickness. Available equipment in the fabricating shops could not deal with material over $\frac{1}{2}$ in. thick and both internal and external cover plates were used to achieve the necessary strength. The method of analysis followed closely on that recommended by Grinter[4].

The side sheeting rails were designed in a similar manner to the purlins, with the exception that the main member was a channel. Tests were carried out to check the resistance to lateral buckling. This was done by erecting the main member in a horizontal position and fixing aluminium corrugated sheets with the hook bolts arranged on alternate sides of the web. The beam was then loaded to simulate the design stresses as shown in Fig. 11 and proved entirely satisfactory to the Ministry of Works. The test was later repeated after alternate hook bolts had been removed and those remaining proved adequate to stabilize the member laterally.

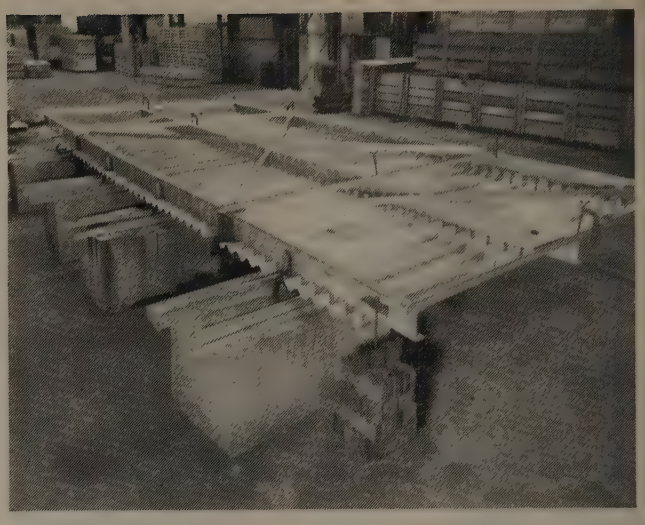

Fig. 11.—Ministry of Works, Thurleigh—Loading Test on Sheeting Rail

The structure was shop riveted, using cold driven rivets and the contact surfaces of all members were treated with zinc chromate to prevent electrolitic action. This precaution is not normally taken with aluminium structures unless different types of alloys are being used or the structure is in a particularly corrosive atmosphere. Aluminium decking was again used for the roof and aluminium sheeting, fire resisting board lining and aluminium foil insulation for the walls.

The hangar is closed by electrically operated sliding doors, each door consisting of eight leaves on four tracks. The doors are framed and clad in aluminium, which material has again been used with fire resisting board for the internal lining. The main door members, if designed as a joist section, would be too deep to be extruded by any press available in this country. They were therefore, extruded as two identical joggled tee sections and were shop riveted to form a joist section. As deflection was the criterion of design the shear was small and a reasonable rivet pitch was used. The door leaves, each being 25 ft. wide, were outside the scope of normal transport and were therefore site assembled.

Tubular Steel as a Structural Material

It is known to the man in the street that the most economical strut is a tube. It is therefore surprising that until recent years this material has been so little used in framed roof construction. Until the advent of welding it was not possible to form an economic joint in a tube and it is only in the last decade that tube manufacturers have made serious efforts to foster its use in building. Although tubes have often been incorporated in roof structures the reason for their use has primarily been on aesthetic grounds rather than economical.

An excellent paper on "Welded Tubular Structures," has given been by Bedford[5] and this, together with a more recent paper by Godfrey[6] "The Application of Tubes to Structures" is worthy of study. The following is a brief résumé on tubular construction :—

Before considering the design of tubular structures it is necessary to consider the materials available. Generally tubes fall into three catagories :—

(i) Small seam welded tubes up to 4½ in. diameter costing from £50 ton.

(ii) Seamless tubes up to 16 in. diameter costing from £65.

(iii) Small square or rectangular tubes ranging from £65 ton. This type of tube is comparatively recent and it is likely that new manufacturing techniques will result in a greater range of tube and some reduction in cost.

A further subdivision can be obtained by using different types of steel and those readily available are Grades 13, 15 and 20 with yield stresses of 13½, 15 and 20 ton/sq. in. respectively. The cost of the higher strength steel is little more than the mild quality.

In general, the method of carrying a load is either by beams in flexure or by braced structures. In the latter case the loads in the members are in compression or tension and on occasions due to load reversal in both.

A tube is the perfect strut because the metal is equally displaced about one point and it follows that it is also the perfect section for bending about two perpendicular axes. In simple bending, tubes are less economic than joists or channels since too much of the area is concentrated in the " web " and too little in the " flange." On the other hand, tubes are more efficient in bending than angles. With regard to plastic design it has been stated that the tube has the advantage of a high shape factor when compared with an R.S.J. This statement is misleading in that the lower the shape factor the more efficient the section in bending, whether plastic or elastic. Thus for a joist section with infinitely thin web the shape factor approaches 1.0 and increases as the web is thickened, culminating at 1.5 when a rectangle is reached. An important advantage of any hollow closed section is that in consequence of its high strength in torsion the maximum bending stress may be worked to without fear of lateral instability and this advantage is important in the design of slender beams.

In practice, possibly the best example of lightly loaded small span beams is the purlin. No matter the section shape, account need not be taken of lateral instability, except for ease of erection and sheeting, as once the sheeting is fixed the member is unlikely to buckle. For small spans when angles would normally be used, tubes are economical partly due to the cheapest grade being used. For larger spans, joist or channel sections would be the more suitable shape. In the case of sheeting rails, an example of two plane bending, tubes particularly rectangular should be considered. In the author's opinion deflection of purlins is not a serious consideration as they are rarely, if ever, fully loaded. The simple device of making purlins continuous over two bays more than halves the deflection. For more heavily loaded members, unless the beams are slender, tubes on their own are uneconomic and if it is desired to use tubes, recourse must be made to latticing.

Tubes are efficient as struts particularly when the slenderness ratio is high. When compared with double angles, the saving in weight is about 22 per cent when the slenderness ratio of the tube is 60, rising to nearly 50 per cent when the slenderness ratio of the tube is 120. For economy it is preferable to eliminate lateral loads and to so arrange the structure that the tube has the same effective length in all directions. Alternatively, rectangular tubes or two or more tubes combined should be used.

The clauses covering tension in the current B.S. 449 do not permit any advantage when using tubes as ties as the same stresses are permitted as for other sections symmetrical about one or more axis. It would, therefore, appear logical to use hot rolled sections as ties combined with tubes as struts, although this is often frowned upon on aesthetic grounds.

The most convenient method of jointing tubes is by welding for which rules are given in B.S. 449. The normal method of fabrication is to set out the tubes on giant tables which are arranged with dowel holes at close centres. Adjustable clamps permit the tubes to be fixed in any position, after which they can be welded. After the first off is so assembled, the clamps are left in position and automatically provide a welding jig. It will be appreciated that site welding being dependent on the weather and requiring a certain amount of equipment for jigging is best avoided. It is customary, save in exceptional circumstances, to construct all site joints with bolts. An additional cost which it is desirable to eliminate is the end shaping of tubes. If the structure is arranged so that shear is a minimum the internal members will be small and end shaping not required. The welding may also be simplified as fillet welds are permitted if the branch tube does not exceed three times the diameter of the main member. If this ratio is not much exceeded, partial flattening is often used to meet this requirement. These methods of jointing eliminate gussets and result in a cleaner and more economical structure. A useful method of simplifying the joint between a number of tubes at a node is to introduce a tube the axis of which is perpendicular to the plane of the joint. The diameter of this tube can be such as to provide ample space for each " spoke " to connect on the circumference without the need for complex end shaping.

Other salient points in tubular construction are that a much thinner section is permissible due to the inside surface being sealed and therefore not susceptible to corrosion. This factor, together with others, enables the dead weight of the structure to be reduced and this results in further economy on large span structures. Of less importance is the saving on painting costs due to the smaller surface area and the elimination of crevices.

Flight Shed for the De Havilland Engine Co.

In 1956 the De Havilland Engine Company decided to proceed with plans to construct a flight shed at Hatfield. In the preliminary scheme the shed was to be 150 ft. deep × 35 ft. high with a length of 270 ft., and it was required to provide for future extension at one end. As the Client required freedom from columns, a cantilever hangar was considered, but as a double cantilever was impractical and the foundations of a single cantilever would have been costly, this was ruled out. Had workshops been required, no doubt the additional dead weight could have been utilised to hold the structure down and the scheme would have merited further consideration. Instead it was decided to construct the shed with two bays and a two way span, thus only one central column would cause an obstruction.

A functional specification was drawn up and selected, contractors invited to tender on a design and construction basis. The loading was to be in accordance with B.S. Code of Practice C.P.3 (Ch. V) Loading, with a wind velocity of 65 m.p.h. Although the superimposed load is higher than that for the Comet Flight Shed on the same airfield, it was considered that the additional load would permit the installation of small runways without serious structural modification. The roof glazing was specified to be 25 per cent of the floor area and in addition a generous amount of glazing was to be incorporated in the North Wall. The remainder of the roof was to be metal decking and the wall construction aluminium sheets, aluminium foil insulation and fire resisting board lining. Tenders were invited from steel and aluminium contractors and the intermixing of these materials was permitted. Consideration was also given to concrete construction, but as the shed was urgently needed and winter construction essential, it was decided against this medium. The lowest tender was submitted by Tubewrights Ltd., and this was accepted on behalf of the Client. After further consideration, the length of the shed was increased to 300 ft. and the clear height to 45 ft.

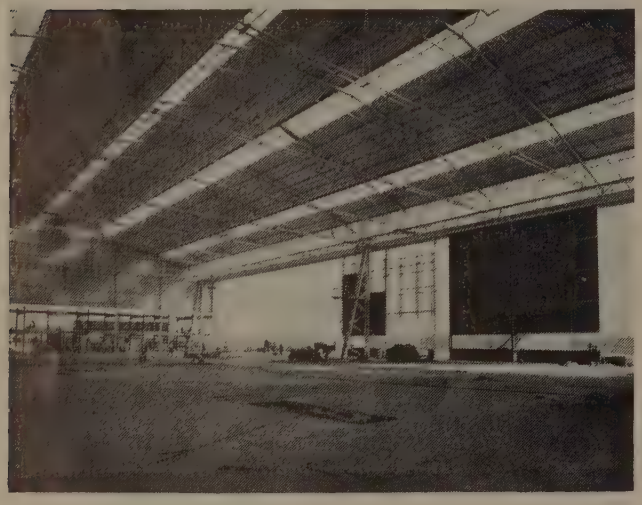

Fig. 12.—De Havilland Engine Co. Flight Shed—Structure Completed and Doors in course of Erection. (Monro & Partners, Architects and Engineers)

A general view of the shed is shown in Fig. 12. The main roof frames span from front to rear and are supported over the doors by a deep lattice girder. By this arrangement, future extension can readily be made and the present structure is not designed for a great deal of additional load from these extensions. The roof frames are designed as 3 pin latticed parabolic arches, the thrust being countered by ties connecting the springings. The position of the pins was so arranged that the main compression was taken by the upper boom of the latticed frame, the inner boom being incorporated to cater for secondary bending. The upper boom consists of a $10\frac{1}{4}$ in. dia. tube and with this exception the remaining members were $4\frac{1}{2}$ in. or less in diameter. The internal members are so arranged that there is considerable repetition in fabricating and due to the small amount of shear the members are small, welding is simplified and end shaping in part eliminated. The tie consists of two tubes which, in the event of wind loads, may be in compression ; they are, therefore, spaced 18 in. apart and supported vertically from the arch. The general shape of the arch is pleasing in appearance and readily lends itself to small modification in the outline of the cladding to accommodate glazing at a safe slope. The purlins are designed as latticed girders and cantilever in alternate bays. By this simple expedient the bending moments and consequently the boom sizes are considerably reduced. Where the top boom supports decking, a tube is used and in the case of glazing a cold formed zed section forms the top boom. Tie rods at third points provide stability to the purlin booms and act also as anti-sag ties.

At the front of the hangar a deep girder supports the arch frames over the door openings and again the cantilever principle is used to reduce bending moments and loads. The West Bay girder is projected one fifth of its span on either end into the adjacent bays as shown in Fig. 13 and thus reduces the span of the East Bay by 20 per cent. Twin tubes are used for both boom and internal members and in the former case lead to economy due to differing lengths in both directions. The main girders were shop assembled as far as transport permitted. On site they were assembled below their final position and site welded before being lifted into position. Lateral stability was provided by tying back to the roof arches immediately below the glazing.

Fig. 13.—De Havilland Engine Co. Flight Shed—Detail at South-West Corner Showing Provision for Future Extension

The general structural requirements of the building did not facilitate the use of portal frames and the columns were therefore cantilevered from the foundations. The main columns were subjected to a considerable wind load and hence they are tapered away from the foundations. Twin tubes battened together were again used in view of the lateral length and light bracing provided stability in the other direction. A ring wind girder was provided around the circumference of the shed at door head level and took account of the wind loads on the remaining columns. The longitudinal wind load is transmitted at the front of the shed by a cigar-shaped raking member and at the rear by

normal wind bracing. The remaining columns were therefore designed simply supported top and bottom and were constructed with latticed tubes. The sheeting rails were constructed in a similar manner to the purlins and because of the large number of bracing intersections it became necessary to position both the sheeting and lining on the outside of the rail. This posed minor problems which were satisfactorily solved by the sheeting contractor.

Another unusual feature of the shed is the main foundations. Due to the large bending moments involved and the need to cater for uplift it proved economical to construct the foundations for the main columns in the shape of joists. This was done by excavating to a joist profile or plan, the size of which was 30 ft. × 15 ft., the web and flanges being 5 ft. thick. The reinforcement was then fixed and the concrete deposited in the normal manner.

One of the major costs involved in the construction of an aircraft hangar is that of the doors. If full openings are required in a multi-bay hangar the solution lays between nesting or folding doors on one or two tracks or alternatively simple leaved doors on a multitude of tracks. In the case under consideration three tracks were used with normal interconnected leaves and this permits two thirds of the hangar length to be opened at any one time. The doors are electrically operated and interconnected in such a manner that by depressing a lever at one end the doors continue in motion " picking up " the adjacent leaves until they are electrically cut out at the end of their travel at the other. A further refinement is that the centre bank of doors can be split and opened to either side of the centre column to permit the passage of small aircraft. The main framework is of mild steel and the cladding is a composite panel construction with steel corrugated sheets on the inside and flat aluminium on the outside, thermal insulation being provided by glass silk between.

Conclusions

From the foregoing description it will be seen that the advantages of aluminium have been harnessed in the construction of some interesting hangars most of which were built as a result of competitive tender. Noteworthy is the use of the extrusion process to obtain section shapes ideally suited to the structural problem. In the case of the Admiralty hangars, the easy handling by virtue of the light weight will be appreciated.

In all the hangars described, an effort has been made to reduce weight and cost to the minimum and to eliminate non-load bearing metal. This reduction in dead weight on the large spans has also resulted in further economy.

In the case of tubular steel, the author feels that the De Havilland Engine Company Flight Shed is one of the forerunners of many hangars in this material. In particular it will be found economic in the construction of large span cantilever hangars when used in conjunction with reinforced concrete.

Acknowledgments

The author wishes to thank G. J. Monro, B.Sc., A.R.I.B.A., A.R.I.A.S., for his constant encouragement in seeking new or improved structural solutions both in hangar construction and in other fields. Thanks are also due to Messrs. Hawksley—S.M.D., Admiralty, Ministry of Works and Messrs. Tubewrights for their permission to publish information and photographs. The design staff who sometimes worked with the author on the aluminium hangars were C. M. Marsh, B.A., A. V. R. Bowen, A.M.I.Mech.E., F. J. Melton, A.M.I.Struct.E., and P. G. Ridley, B.Sc., A.M.I.Struct.E. The designer of the tubular steel hangar was F. E. S. West, M.M., A.M.I.Struct.E., and J. Bak, B.Sc., M.I.Struct.E., A.M.I.C.E., was the consulting engineer for the foundations of the Comet Flight Shed.

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The Use of Arc Welding in Structural Engineering*

By J. S. Allen, A.M.I.C.E.

Introduction

The structural industry was one of the early users of electric arc welding and there are many examples which date back to the 1920's. Its use increased steadily until the Second World War, but it was during the war and in this post war period that the greatest increase in its application has taken place. It has now become one of the leading methods of fabrication and welded work, as a percentage of the total structural production in this country, rivals all other methods. Indeed, in many structural fields, for example rigid frames and structures designed by the plastic theory, it is now the only method of fabrication in use.

Recently, the Structural Committee of the Institute of Welding decided to submit to the structural industry a questionnaire, the replies to which showed that approximately 40 per cent of all output of the firms who replied to the questionnaire is welded and that this percentage is increasing. The report of the Committee indicated, however, that there is a shortage of designers and detailers capable of producing sound welded design and details.

An extract of this report is given below :

" Since the war, equipment and technique have so improved that, in the shops at least, welding provides a reliable alternative to almost every riveted job, and can be used in cases where riveting would be impracticable. Manipulators and automatic welding machines have been built to deal with pieces as large as one hundred tons in weight and one hundred feet in length. Pieces of half this size are becoming a matter of common practice. On the metallurgical side, research and development have led to the production of electrodes suitable for welding in all positions most grades of steel used for constructional purposes. In the field of weld examination, whereas the practice of visual inspection and mechanical testing of samples is almost universal practice in the Industry, the examination of welds by X-ray, gamma-ray and ultrasonic equipment is responsible for a steady growth of confidence in the successful application of welding to an ever-widening range of fabrications.

"In riveted work a limit has been reached to the number of possible structural forms and shapes ; in welded work not only can this range be covered just as effectively but new forms are developing and will continue to develop as the particular advantages of welding are exploited.

"Welding has broadened the approach to structural design ; the portal frame affording maximum uninterrupted space beneath it, and the Vierendeel girder, providing clear openings for windows, are typical examples of rigid frames for which welded construction is particularly suitable.

"Throughout the whole range of structures, from building frames to bridges, the simplicity of detail and clean lines and surfaces, resulting from the less restricted use of component sections and from absence of rivet heads, gives welded construction an appreciable advantage from the point of view of maintenance and also a character of considerable aesthetic value . . . . "

Most types of structure can be welded with advantage, but to obtain the greatest advantage, the designer must base his design on the principles required by welding. It is most unsatisfactory to think in terms of a riveted structure and to substitute an amount of weld equal to the strength of the rivets which would normally be used.

Welding produces rigid joints and the solid or rigid type of frame is usually the most satisfactory. Sometimes the structure designed in this way will have a greater overall weight than the conventional riveted design as for example, the use of a portal frame in lieu of a trussed frame, but the overall cost should show the advantage with the welded frame.

Types of Structures which can be fabricated by Welding

It is now intended to discuss briefly the various types of welded structure and their relative merits.

(a) *Roof Trusses and Light Angle Frames*

The roof truss is one of the most widely used types of roof structure, although there is an increase in the popularity of the portal frame. The roof truss is a light structure which it is very simple to design and little special equipment is needed for its construction. It is possible for the lighter type of fabrication shop to produce fully satisfactory trusses for use in factory buildings, storage sheds and the lighter type of structure.

Certain advantages are to be obtained by designing trusses as welded structures rather than the more conventional riveted design. These are as follows :

i For simple connections, gusset plates are not usually required and therefore a saving in the cost of material is obtained. Gussets in riveted work are usually provided to enable sufficient rivets to be put in a connection.

ii The section of tension members may be reduced, as there is no deduction in the section for rivet holes. Hence a further saving in weight can be obtained.

iii It is possible to use smaller sections than those which could be used in a riveted design, as the question of rivet back marks and edge distances does not apply. Within limits the sections may also be made thinner.

* *Paper read before the Midland Counties Branch of the Institution of Structural Engineers at Wolverhampton on February 6th,* 1957 *for which the Author was awarded the Midland Counties Branch Prize for the Session* 1956-7.

iv The welded joint can be completely sealed by welding all round the mating edges and thus there is less probability of corrosion attack than with the riveted joint.

v Some saving in fabrication cost in the marking off and drilling of members is obtained.

Against the saving in weight of materials used, must be balanced possible higher fabrication costs.

It is usual, for small quantity production, to set out the truss with chalk lines on the shop floor, or preferably on a large welding table. The members are laid in position and tack-welded together, and owing to the flexibility of a truss, it is preferable to weld as much as can be completed before moving the truss and turning it over for completion of the welding.

To assemble a roof truss accurately without some form of jig is a difficult operation. If a sufficient number of similar trusses is required, the provision of a jig should be economical, but for small quantities the cost of a jig may outweigh the other savings. So-called simple jigs are liable to be quite expensive, particularly if the trusses being produced are not likely to be repeated.

Frequently trusses designed for welding have single hole connections provided to enable the truss to be accurately assembled before welding, the holes being subsequently welded up. This procedure for small quantities of the larger trusses is considered by some to be essential in lieu of jigging. It will be appreciated that this handling for marking and drilling and subsequent welding up of holes, may in certain circumstances make this type of welded structure a doubtful proposition, but it is economical in many cases.

There is often no necessity to weld up the holes as will be seen from Fig. 1, which shows the result of an experiment where a simple connection was tested in a tensile machine. The specimen comprised a welded joint between two 2 in. by $\frac{1}{4}$ in. flats through which an $\frac{11}{16}$ in. dia. hole was drilled, usual tacking holes being about $\frac{5}{16}$ in. diameter. The joint failed in the parent metal away from the hole, showing that the hole did not affect the ultimate strength of the joint.

It is felt that although the advantages noted in the foregoing may be obtained by welding, the truss and latticed structures are essentially riveted designs and the designer should consider the merits of a portal frame and rigid structures before adopting this type of design.

Fig. 1

Overwelding is a danger to be avoided in light angle members, as it can cause distortions which are difficult to prevent or correct. A recently experienced example concerned glazing purlins which comprised 5 in. by 3 in. main angles with $1\frac{1}{2}$ in. by $1\frac{1}{2}$ in. glazing angles connected to them by batten plates. The connections were fully welded with the result shown in Fig. 2.

The connections needed an insignificant amount of fillet weld to carry the design load and when subsequent members were welded with short intermittent welds, no appreciable distortion occurred.

Fig. 2

(b) *Columns and Struts*

Many compound sections can be designed which are suitable for welding. These sections may comprise standard rolled sections or fabricated plates, or a combination of the two. It is rarely possible to obtain an ideal section from standard rolled sections, as they are limited to shape and size and more satisfactory sections can often be obtained by the use of fabricated plates.

Some sections suitable for welding are shown in Fig. 3 and on many of these automatic welding can be successfully applied.

Box sections are a true welded design providing the maximum rigidity and savings of weight of up to 20 per cent on an equivalent three plate I section member can be obtained. This section of column is becoming increasingly popular and is being adopted in many new power stations.

(c) *Beams and Beam Connections*

Economy can be obtained if beams are designed with rigid or semi-rigid end connections and these conditions can easily be obtained by welding. Fig. 4 shows four types of rigid connection, two being to a single shaft column and two to a plated column.

(d) *Plate Girders*

Plate girders for maximum efficiency should be of I or box section. There are many advantages to be gained by the adoption of welding in girder work, the main ones being in the full utilisation of the sections,

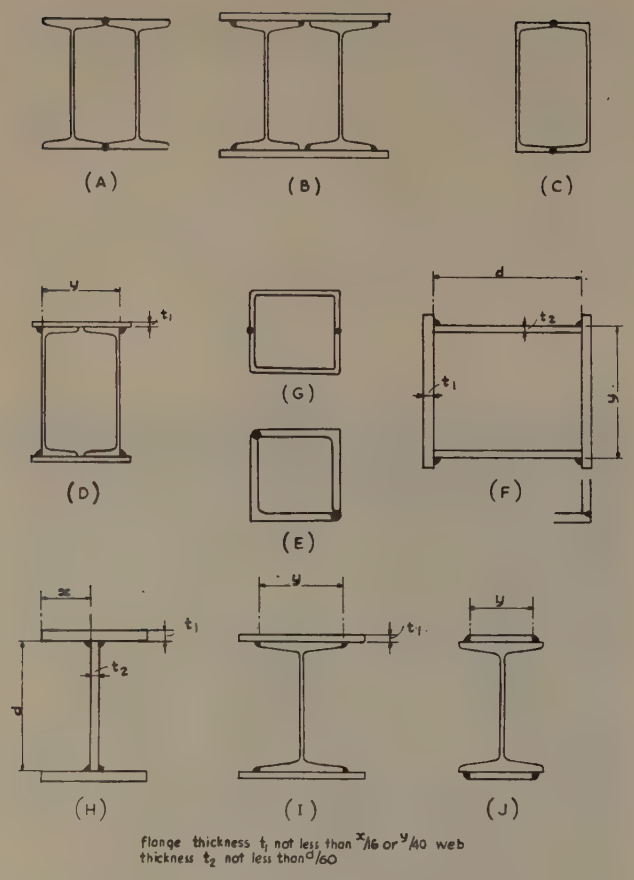


Fig. 3

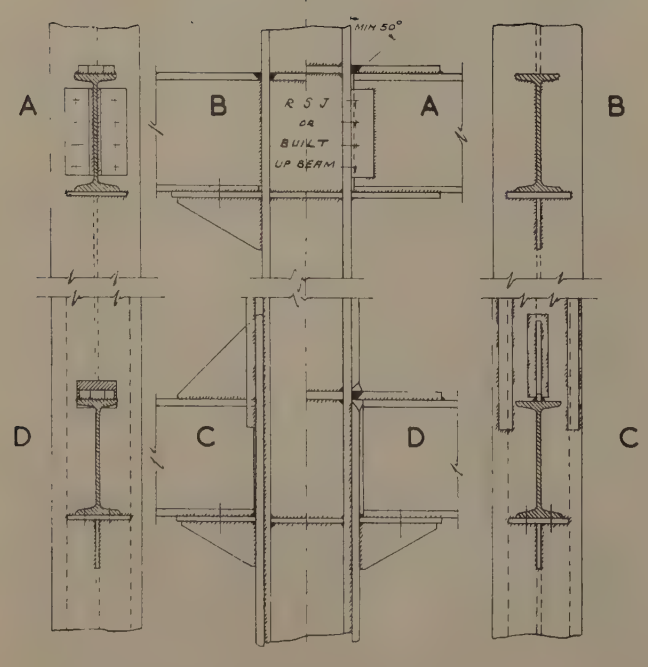


Fig. 4.—Beam to Column Connections. Fixed Jointed Type

due to there being no rivet holes and in the omission of connecting angles between the web and flanges. Weight savings can be obtained by the use of flats as stiffeners or of angles welded at their toes. The joints in the webs and flanges will develop the full strength of the sections and no cover plates are necessary, again saving weight.

If these members are welded, a saving in weight of material of about 15 per cent can be made, together with a saving in fabrication cost. Other benefits include a clean appearance and ease of maintenance.

It will be seen that a welded plate girder will usually prove a more attractive proposition than a riveted girder. However, bad design which is more serious in welding than in riveting, can increase the cost of fabrication to outweigh the savings in material. The most economical design will usually prove to be the one using the least amount of welding compatible with the minimum weight and this should be the designer's aim.

(e) *Rigid Frames*

Rigid frames rely on the rigidity of their joints and are thus an ideal type of construction for welding. Rigid frames can, and have been, constructed in riveted design, but the number of rivets used and the complexity of the joints so designed make this type of construction an uneconomical proposition.

Fig. 5 shows a comparison between the riveted and welded knee in a portal frame.

The portal frame is gaining increasing popularity. There are numerous types of frame and these may comprise a combination of rigid and theoretically pinned joints. It is important from the welding point of view, that the practical aspects of the details and sections of the various members should be appreciated.

A portal frame if it is properly designed and detailed will, due to the full utilisation of the steel sections, be less wasteful of material and will involve less template making and setting out, than the corresponding trussed design and should not show too great an increase in material consumption.

It follows that portal frames should show very little, if any, increase in the initial cost over the trussed design and that when the indirect savings are considered, portal frames should undoubtedly show considerable overall saving. These indirect savings include for approximately equal weights of steelwork :-

i Cleanliness of line.
ii Less area of steelwork to be painted.
iii Gains in crane clearance due to utilisation of roof space.
iv Gain in clear width between stanchions.
v Reduction in height to eaves level.
vi Foundations which are required to withstand direct forces only.
vii Less maintenance costs.
viii Less welding.
ix Less area for dust lodgement.

These points are brought out in Fig. 6.

Site Welding

There are several differences between the application of welding in the shops and on site.

In the shops, facilities are usually available to handle members, such that most of the welds can be made in the downhand position which is the most economical,

Fig. 5

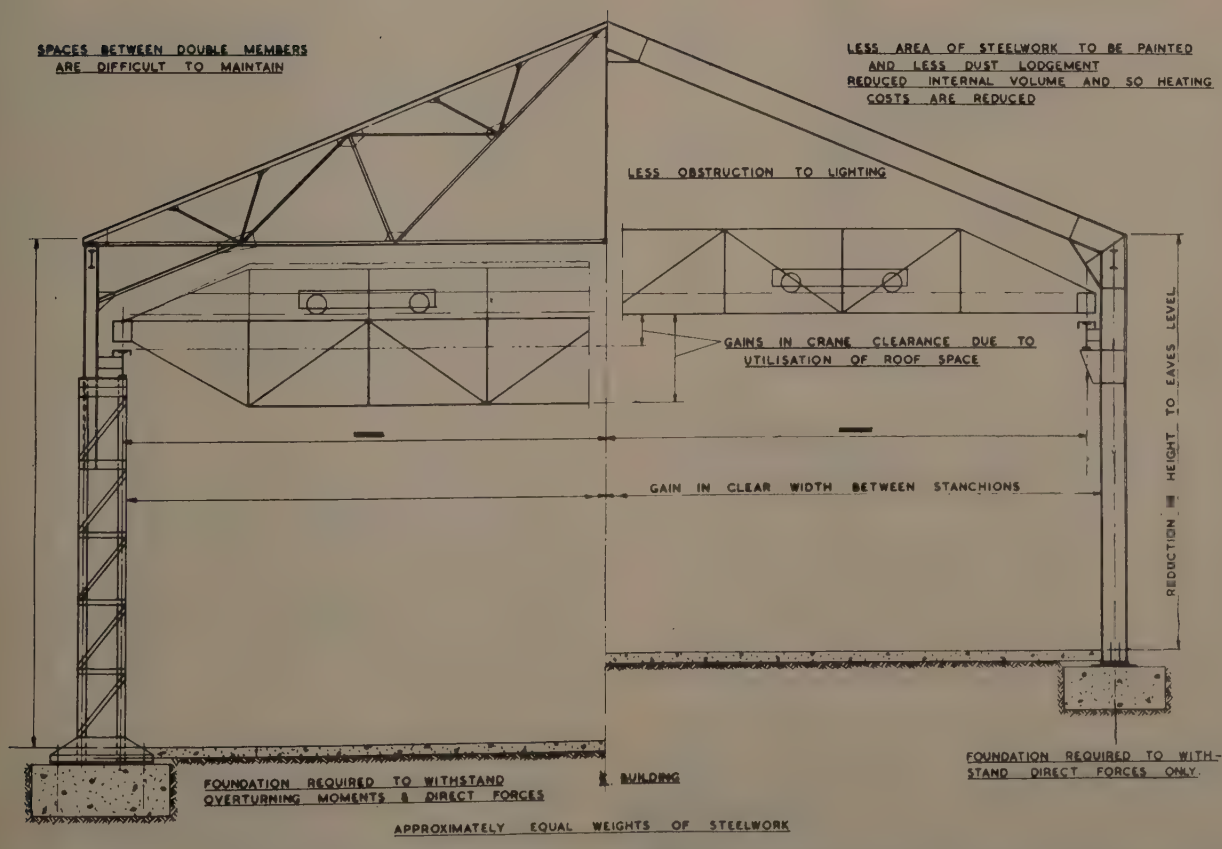

SPACES BETWEEN DOUBLE MEMBERS ARE DIFFICULT TO MAINTAIN
LESS AREA OF STEELWORK TO BE PAINTED AND LESS DUST LODGEMENT REDUCED INTERNAL VOLUME AND SO HEATING COSTS ARE REDUCED
LESS OBSTRUCTION TO LIGHTING
GAINS IN CRANE CLEARANCE DUE TO UTILISATION OF ROOF SPACE
GAIN IN CLEAR WIDTH BETWEEN STANCHIONS
REDUCTION IN HEIGHT TO EAVES LEVEL
FOUNDATION REQUIRED TO WITHSTAND OVERTURNING MOMENTS & DIRECT FORCES
½ BUILDING
FOUNDATION REQUIRED TO WITHSTAND DIRECT FORCES ONLY.
APPROXIMATELY EQUAL WEIGHTS OF STEELWORK

Fig. 6.—Items of Comparison between Standard Truss and Two Hinged Portal Frame Construction for a typical 60 feet Span Factory Building with E.O.T.C. Gantry

whilst on site most of the work will have to be carried out in position, often at considerable height from the floor and scaffolding may be needed for the welders.

In the shops ample welding plant capacity is available, whilst at site the size of electrodes which can be used is often limited by the output of the available plant. The majority of mobile plants in general use are of about 300-350 amp capacity. A further factor is the distance of the work from the welding plant and on site this may be excessive, thus reducing the available current by drop in the cables.

The weather may seriously affect progress on site and the provision of suitable shelters and screens may have to be considered.

It will be seen that the physical cost of welding a particular joint on site will be higher than that of the same joint in the shops, but often the site overheads will be considerably less than those obtaining in the shops and the total costs will not be too disproportionate.

In many companies, site welding is avoided, but this should not be so, as there are many cases where it can be used to advantage. A number of these cases are listed below.

1. *New Structures*

(a) To obtain full continuity as required by design.
(b) To assemble parts into continuous lengths which would be too long to transport, or into larger pieces which would be too large to transport.
(c) To join components where the reverse face of one of them is inaccessible.
(d) To obtain watertightness.
(e) To obtain a flush surface and clean appearance (whether for aesthetic reasons or for ease of laying waterproofing, floorings, etc.)
(f) To fabricate at site, with less elaborate drawings, such secondary structures as stairways, platforms, railings, racks, etc.
(g) To fabricate a structure which could not be assembled in any other way (e.g. structures designed by the plastic theory).

2. *Existing Structures*

(a) To strengthen.
(b) To alter, repair or extend.

One of the difficulties often quoted against site welding is the lack of suitable supervision. This is quite a problem as the site supervisor may not be experienced in welding and it is obviously not economical to have a welding supervisor on site in control of a small number of welders. The problem may be overcome by the appointment of a Site Welding Engineer, whose duty it is to visit at frequent intervals all the Company's sites upon which welding is being used. The knowledge that the work is to be inspected frequently will have its effect upon the quality of the work.

The use of site welding for small jobs and simple joints is often questioned, but when the economics are being considered the job as a whole must be taken into account and the shop savings in marking out, drilling, etc., included.

From the site point of view, welding equipment and ancillary plant must be considered as normal equipment. It is useless to expect to erect a welded job with tackle suitable for riveted work. The tackle must be of the type suitable for welded joints and ample supplies of suitable clamps and packs must be available.

One large Structural Company which specialises in the lighter type of structures such as school buildings, etc., makes extensive use of 'Lindapters' to make temporary connections and thus avoid drilling. In conjunction with these 'Lindapters,' a series of clamp plates and bolts are used. It is thus possible to erect and site weld this lighter type of structure without temporary fixed cleats and bolts.

A further method used on this lighter type of structure is the assembly and welding of frames on the floor, these frames being subsequently lifted in one section by crane.

It will be of interest to note that the Institute of Welding has appointed a panel which is at present engaged in the preparation of a booklet on Site Assembly and Welding, and this booklet should be available in the course of a few months.

Inspection

With hand welding there is the ever present human factor and although it is possible to produce first class welds with good penetration, no undercutting, or slag inclusion, good profile and of correct size, these matters are largely in the hands of the welder and he being human, will like all of us have human failings which will prevent 100 per cent results being obtained continuously.

Some form of inspection is therefore necessary and for structural work the most satisfactory and cheapest method of inspection is in the employment of a good inspector during the time of welding. He will be able to see the individual runs being laid and will be in a position to have the work stopped if unsatisfactory techniques are being employed. He will also have the advantage of knowing the welders on the job personally and will have his own ideas of their individual capabilities and trustworthiness.

It is always more difficult and more expensive to repair a weld fault satisfactorily than to carry out a satisfactory weld initially. It is obvious, therefore, that the supervision and control should be concentrated on the checking of preparations and assembly and in the actual deposition of the weld metal.

There are, of course, methods of weld testing both destructive and non-destructive.

1. Destructive

(a) *Butt Welds*

Specimens of butt welds can be cut up and made into test specimens which in the Laboratory can give tensile, bend, reverse bend, nick break, all weld, izod or charpy tests. With normal care, correct preparation and average welding skill, it is quite simple to deposit welds on mild steel which will give satisfactory results for each of these tests.

(b) *Fillet Welds*

Fillet welds can be tested by breaking a specimen to show the root penetration or a test can be prepared which will test the shear strength of the welds.

All the above tests must be made from specially prepared coupon plates, which are usually attached to the job.

The tests add to the cost of the job and should be kept to a minimum. They are useful for the testing of welders to determine their capabilities.

It is an ever increasing practice for the customer or his representative to demand that the welders who are to be employed upon a contract shall be tested in accordance with some suitable specification.

This may mean that a welder may have to carry out a similar type of test many times during a year's work. This is both wasteful of time, money and material. It is suggested that there should be some generally accepted authority by which welders could be tested, say yearly.

In the writer's recent experience, a customer would not allow the use of normal rutile electrodes supplied by a manufacturer of world wide repute whose electrodes are tested by Lloyds Register of Shipping until these had been tested to a most stringent specification of their own. These tests were required for each gauge of electrode used. The contract although of fair size, consisted entirely of $\frac{1}{4}$ in. fillet welds, many of which were very lightly stressed. The specification of such tests adds greatly to the cost of welding and may react adversely against its adoption.

2. Non-Destructive

Various methods are available for the non-destructive testing of welds, but most of these are more suitable for testing butt welds.

The principal method is x-ray, whilst later developments use radio-active isotopes to inspect by gamma radiation. In these techniques, a sensitive film is placed in a casette or holder behind the weld to be examined. The X or gamma rays pass through the weld and expose the film.

Thick portions of the object will absorb more radiation, thinner sections less, so that the film will be affected according to variations in object thickness. Blow holes, porosity, cracks, etc., will therefore appear on the film as areas darker than the image produced by the surrounding sound parts of the object.

A later technique known as ultrasonic testing is now being introduced for certain applications, but here a very great deal depends upon the operator and his interpretation of the effects visible upon the cathode ray tube screen. Further it is not readily possible to obtain any permanent record of the results obtained.

The foregoing testing methods are expensive and will directly add to the cost of the job in themselves and indirectly by time delays.

In structural work it is felt that for static loads, these non-destructive tests are not warranted. For this work visual inspection during welding should be sufficient as the high factors of safety employed are sufficient to avoid failure of a weld even if it be of comparatively poor quality. For dynamic loading where fatigue failure is a possibility or in structures which may be subject to brittle fracture conditions through low ambient temperatures, a good case can be made for their use.

In all cases the importance of the joint under consideration should decide the type of inspection to be employed.

With regard to the interpretation of X-ray negatives and gammagraphs, there is no generally accepted standard of weld quality laid down for structural work and unfortunately, when these methods are employed on structural work, inspectors tend to demand standards comparable with Lloyds Class 1. For static loads this standard is higher than necessary and many physical tests have been satisfactorily carried out on welds which included considerable amounts of porosity and slag inclusions.

It is appreciated that the formulation of a standard is a difficult task, as except in the case of a 100 per cent clear film, the results of any two welds will not be exactly alike. It is felt however that inspectors should have considerably more training in the interpretation of radiographs and particularly in the knowledge of the effects of various minor weld faults.

Welding Procedure, Distortion Problems and Correction Methods

1. *Design to avoid Distortion Troubles*

Wherever and however heat is applied to mild steel, contraction will occur. It is not possible to prevent this, but the contraction can be controlled. In welding, distortion is caused by the heat generated by the arc. The general effect of a length of weld is to contract along its length and also across its cross-section. If these contractions are resisted, stresses will be set up in the welds and adjacent material, and these stresses are known as residual stresses. It is therefore desirable to control the distortion and sometimes subsequently relieve the residual stresses by means of mechanical or heat treatment. Types of distortion encountered in a butt joint are shown in Fig. 7.

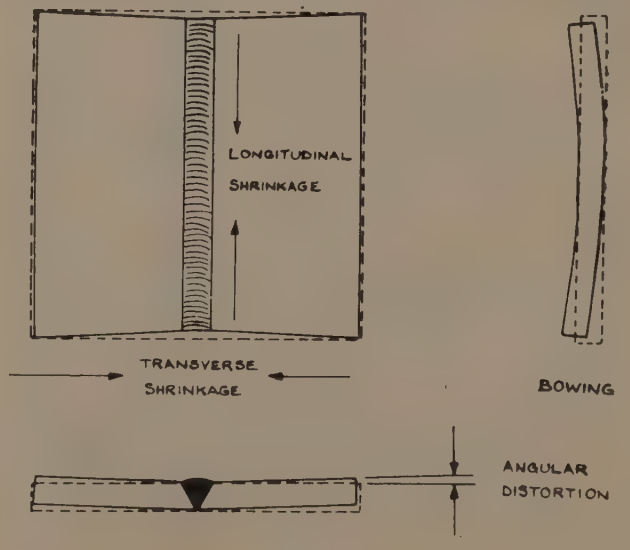


Fig. 7

If the welding of a butt joint is considered, it will be seen that the first root run will tend to pull the plates together and as they are free to move, little stress will be induced. The second run of weld will try to do the same, but this run will be restrained by the run beneath it and this ' under-run ' will have to be compressed before the plates can move any closer together. This pull at the top and push at the bottom of the weld gives rise to an angular distortion. In a heavy butt weld with a number of superimposed runs

all trying to contract together, plus the initial contraction of the root run, there will be a transverse contraction in the joint which will be in the order of $\frac{1}{16}$ in. Fig. 8 illustrates this action.

The total amount of contraction is a function primarily of the first run and secondly, of the number of subsequent runs. It is practically independent of the plate thickness, so that it will be seen that to reduce this contraction, the largest practical gauge of electrode should be used to deposit the minimum number of runs.

Similar forces to those described will exist along the length of the joint and will produce longitudinal contraction and in some cases bowing. This contraction is smaller than the transverse contraction and is approximately $\frac{1}{32}$ in. per 10 ft. of weld.

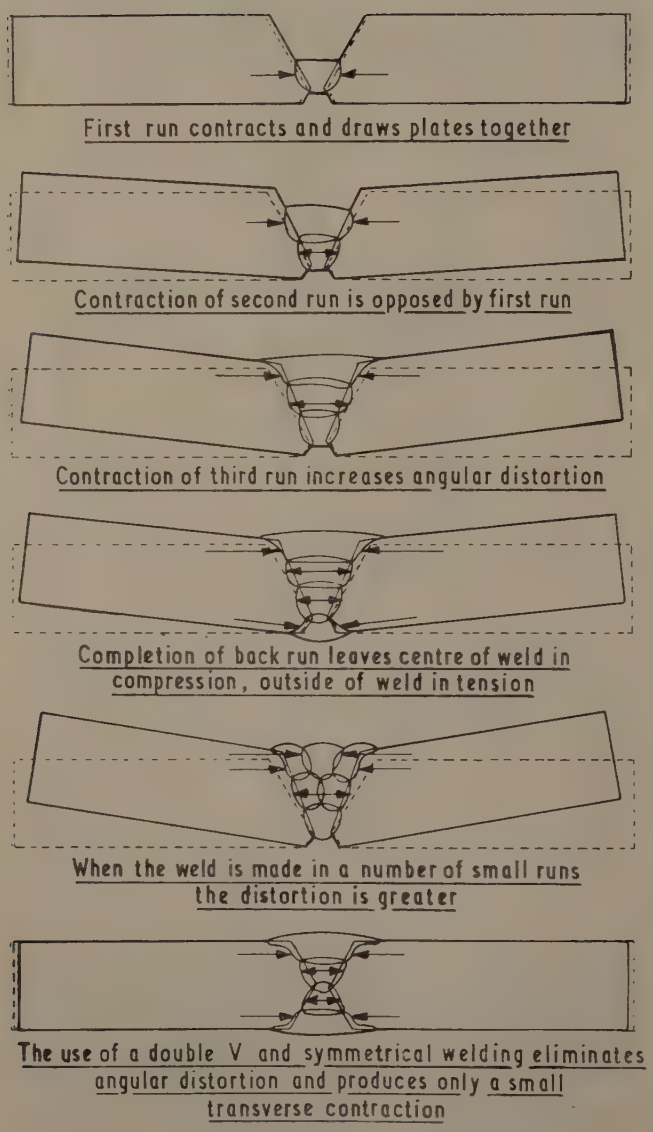

First run contracts and draws plates together
Contraction of second run is opposed by first run
Contraction of third run increases angular distortion
Completion of back run leaves centre of weld in compression, outside of weld in tension
When the weld is made in a number of small runs the distortion is greater
The use of a double V and symmetrical welding eliminates angular distortion and produces only a small transverse contraction

Fig. 8

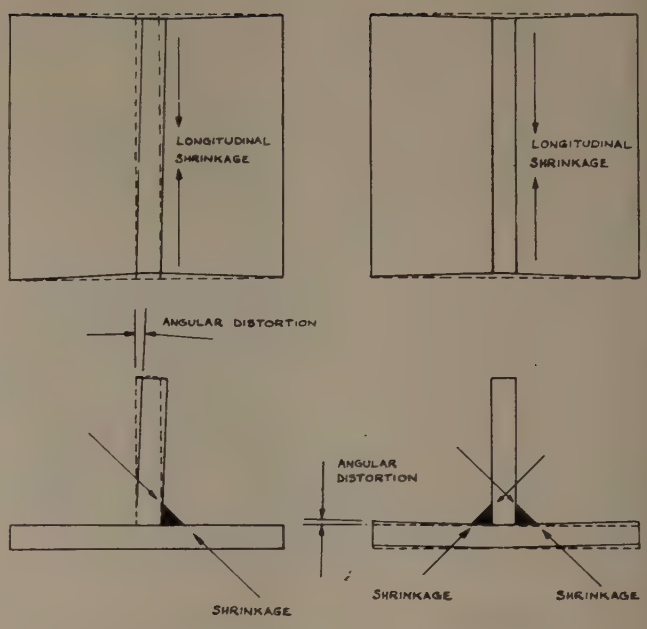
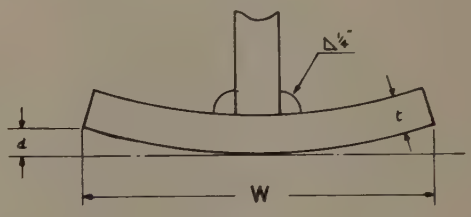


Fig. 9

Contractions produced by fillet welds are shown in Fig. 9.

For fillet welds the transverse contraction is approximately $\frac{1}{32}$ in. providing that the leg length does not exceed $\frac{3}{4}$ of the plate thickness. Fig. 10 gives a table of distortions in flanges produced by $\frac{1}{4}$ in. fillet welds with factors for other sized fillets.



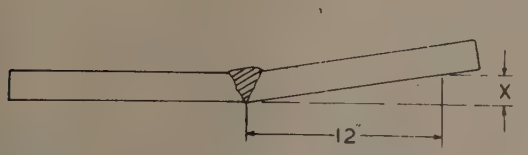
WIDTH W	THICKNESS t INS.							
	3/8	1/2	3/4	1	1 1/4	1 1/2	2	2 1/2
6	3	2	1 1/2	1	1			
9	6	5	3	2	2	2	1	1
12	9	7	5	3	2	2	2	1
15	12	9	6	5	4	3	2	2
18	15	11	7	5	4	4	3	2
21	18	14	9	7	6	5	3	3
24	21	16	10	8	7	6	4	3

DISTORTION d IN 1/64 INS WITH 1/4" F.W.

MULTIPLY BY FACTORS — 3/8" — 1·15; 1/2" — 1·5; 3/4" — 1·8 FOR OTHER FILET SIZES

Fig. 10

Fig. 11 shows a table giving distortions in a completely unrestrained joint related to the number of runs applied. This table shows that the distortion is a function of the number of runs applied and not the plate thickness although of course the two are normally related. It should be noted that if the plates being joined were longer than shown and possessed a reasonable own weight to act as a restraint, these figures would not be as high as shown.



No. OF RUNS	X	No. OF RUNS	X	No. OF RUNS	X
1	5/16	11	15/16	21	1 9/16
2	3/8	12	1	22	1 5/8
3	7/16	13	1 1/16	23	1 11/16
4	1/2	14	1 1/8	24	1 3/4
5	9/16	15	1 3/16	25	1 13/16
6	5/8	16	1 1/4	26	1 7/8
7	11/16	17	1 5/16	27	1 15/16
8	3/4	18	1 3/8	28	2
9	13/16	19	1 7/16	29	2 1/16
10	7/8	20	1 1/2	30	2 1/8

DISTORTION DOES NOT DEPEND ON THICKNESS

Fig. 11

As far as possible, parts of the structure should be allowed to contract freely and so reduce the formation of residual stresses. It is dangerous to restrain completely especially on the thicker sections. If parts cannot be allowed to contract, they must be firmly tack welded and if possible jigged and a welding procedure used which will, as far as possible, reduce the distortion stresses.

A great deal can be done at the design stage to so detail the parts of the structure that the welds are balanced about the neutral axis of the members, thus reducing the distortion. For example, a built up I section will distort less than a built-up T section. The contracting welds will make a length of T section bow and this distortion will be difficult to prevent. The distortion of the I section will be easier to control because the bottom welds cause the girder to distort in the opposite direction to the top welds. This is shown in Fig. 12.

In theory it requires approximately two x lengths of welding on one side of the neutral axis to overcome the distortion of x length of welding on the other side, but in practice members having welds balanced about their neutral axis are found to be commercially straight after welding. If members have to be made in which the welds are not balanced about their neutral axis, it is usual to preset the members by clamping them back to back with a packing piece at their centre.

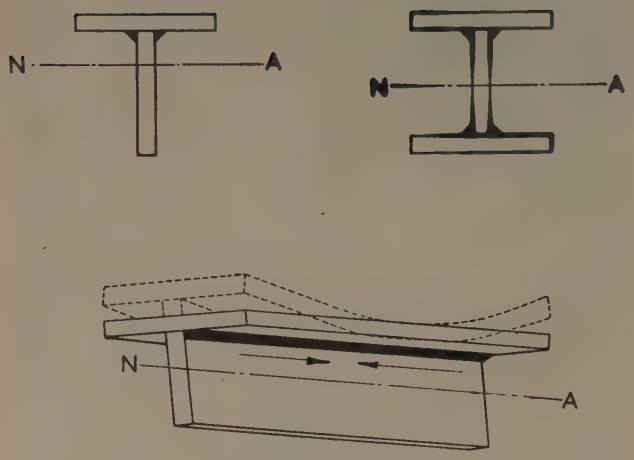


Fig. 12.—Distortion of Welded T Section

The parts are then welded and allowed to cool and subsequently unclamped. It will be found that if the correct amount of preset has been applied the members will spring straight. The amount of packing to be used can only be decided by experience and if the welds are heavy, the thickness may be quite large.

2. *Control of Distortion during Welding*

Much can be done to control distortion during welding by applying a correct welding procedure. For example, long lengths of weld should be commenced at their centre, the welding proceeding towards the free ends. There are two methods of welding which may be employed to reduce distortion and these are known as the "step-back" method and the "doubling-up" procedure.

The "step-back" method of welding, although it may reduce the amount of distortion, is theoretically wrong as welding should always proceed towards a free end and if this is not done, under certain conditions, weld cracking may take place. This method is shown in Fig. 13.

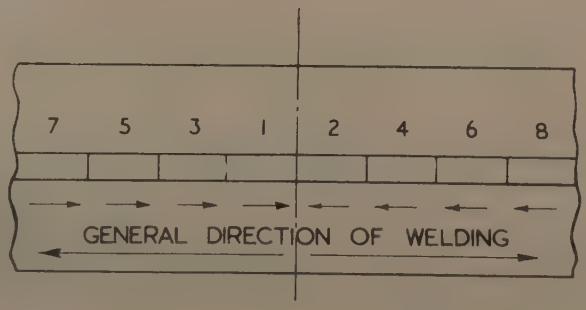


Fig. 13.—Step Back Method of Welding

The fact that it is found that two x lengths of weld is required to balance the distortion caused by x length of weld, gives rise to the doubling-up procedure and this procedure, although minimising distortion, leaves an unwelded portion on one side of the joint when the other side has been completed as seen in Fig. 14.

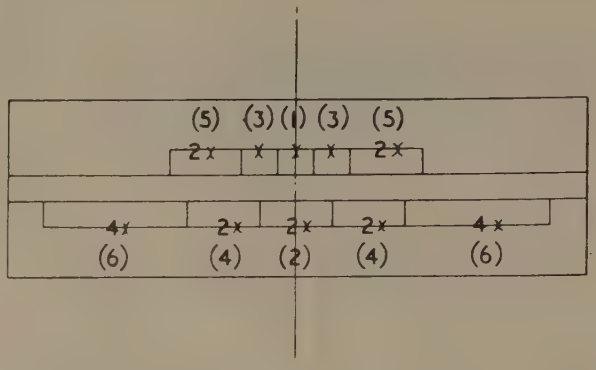


Fig. 14.—Doubling Up Procedure to Reduce Distortion

To avoid this a modified doubling-up procedure may be used as seen in Fig. 15.

A length of weld x is deposited on one side followed by three x on the other and then two x on the first side. With this method distortions are reduced as far as possible, while the same length of welding is deposited on either side of the joint.

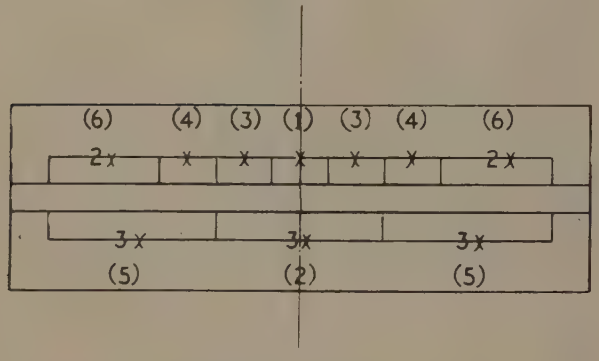


Fig. 15.—Modified Doubling Up Procedure

A schedule showing the order in which welds are to be made should always be provided for any complicated type of work. This schedule will enable the work to be controlled and may have to be modified after completion of the first one off by the results obtained. In cases where no movement can be permitted, the welding procedure must be very carefully thought out and if a considerable amount of restraint has been applied, the job should be subsequently stress relieved.

The designer should bear in mind that the weld sizes should be kept to the minimum and made no larger than necessary for the design strength or the minimum size permitted by metallurgical considerations. By increasing the size of a $\frac{1}{4}$ in. fillet weld to $\frac{5}{16}$ in., the strength is increased by 25 per cent, but the volume of weld metal is increased by 56 per cent. This is due to the strength being proportional to the size of the weld and the volume or cost of the weld being proportional to the square of the size. If the weld sizes are increased too haphazardly, excessive welds may be obtained which, due to the high contraction stresses set up, may crack.

To summarize on the avoidance and control of distortion it can be said that the designer must take into account the following points :

1. The allowance of contraction for welds.
2. The proportion of the joint for minimum weld metal consistent with overall cost.
3. The amount of fixing required.
4. The welding procedure and sequence which will allow the most freedom of movement in the parts being welded.
5. The use of the largest electrodes to give the quickest deposition rates and the largest single run deposits.

3. *Methods of Correcting Distortion*

It frequently occurs that although every endeavour has been made to prevent distortion taking place, members may have to be straightened. There are various methods available for this work, the most universal type of machine being known as a ' Bar or Plate Straightener '. These machines allow a member to pass between their jaws and to bend or straighten the member as required.

If the job is such that it cannot be passed through a machine, a great deal can be done to correct the distortion by applying further heat by means of an oxy-acetylene torch. If heat is applied locally to a member, the heated area will expand, but will be resisted by the surrounding area of cold metal whose strength will be much higher than the heated area.

The metal in the heated area will then be forced to compress itself plastically, so that upon cooling the metal in this area will be of less volume than before heating and so will cause the member to curve in the direction required.

The application of heat should be carefully controlled and considerable experience is required before it can be applied successfully. This method of heat application can also be applied to straighten long strips of plate which have been cut by oxy-acetylene flame along their one edge and due to the release of the internal residual rolling stresses and the effect of the heat of the cut, have curved after cutting. The heat should be applied in triangular areas on the edge opposite the flame cut edges and this is illustrated in Fig. 16.

4. *Various Assembly Procedures*

As far as possible, all constructions should be made in sub-assemblies so that each section can be individually controlled and straightened before assembly into the final structure. For example, if a plate girder is to be constructed using different sections of flange plate and a web made from two or more lengths of plate, the flanges and webs should be butt welded together first as independent units. This procedure avoids

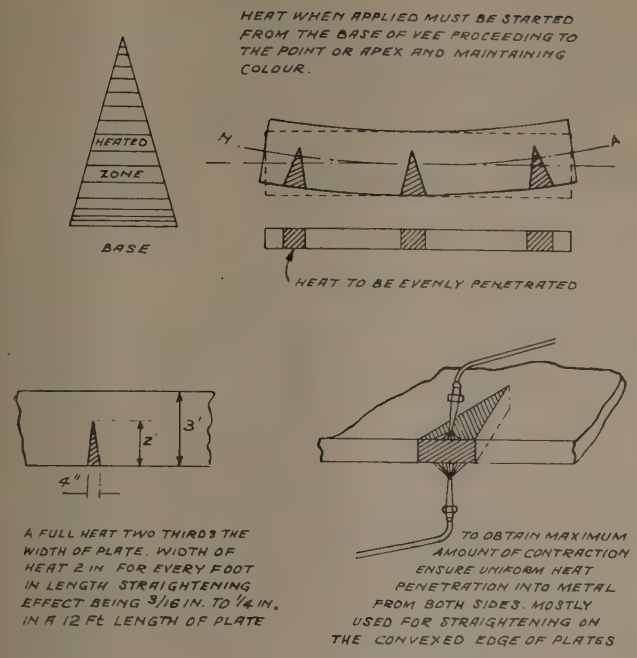

HEAT WHEN APPLIED MUST BE STARTED FROM THE BASE OF VEE PROCEEDING TO THE POINT OR APEX AND MAINTAINING COLOUR.
HEATED ZONE
BASE
HEAT TO BE EVENLY PENETRATED
A FULL HEAT TWO THIRDS THE WIDTH OF PLATE. WIDTH OF HEAT 2 IN FOR EVERY FOOT IN LENGTH STRAIGHTENING EFFECT BEING 3/16 IN. TO 1/4 IN. IN A 12 FT LENGTH OF PLATE
TO OBTAIN MAXIMUM AMOUNT OF CONTRACTION ENSURE UNIFORM HEAT PENETRATION INTO METAL FROM BOTH SIDES. MOSTLY USED FOR STRAIGHTENING ON THE CONVEXED EDGE OF PLATES

Fig. 16

distortion due to the contractions of these main butt welds and as usually the main fillet welds and stiffener welds are balanced, the resulting girder is free from bend or twist.

In the construction of plated portal frames, the column legs can be constructed in two, or three units, for example, the knee, the centre leg section and the bottom leg section. These three sections can then be assembled into their complete length and welded on a suitable level platform or jig.

It is now intended to discuss briefly the weld build-up for a structural site welded joint and the example taken is a fabricated beam splice or a portal column leg to a rafter joint. To specify a suitable weld build-up for a given joint is a straightforward task and may be found by reference to booklets issued by electrode manufacturers, or from weld build-up charts prepared by individual companies.

However, there are various other factors which must be taken into account when specifying the full details of a welded joint. These are to some extent the result of experience with given types of joint, but once the principles are understood they can be applied to other similar joints.

The joint shown in Fig. 17 may be a site joint in a welded beam, or in a portal frame column section to rafter connection. In the latter case the section might be at an angle to the horizontal of up to 30°.

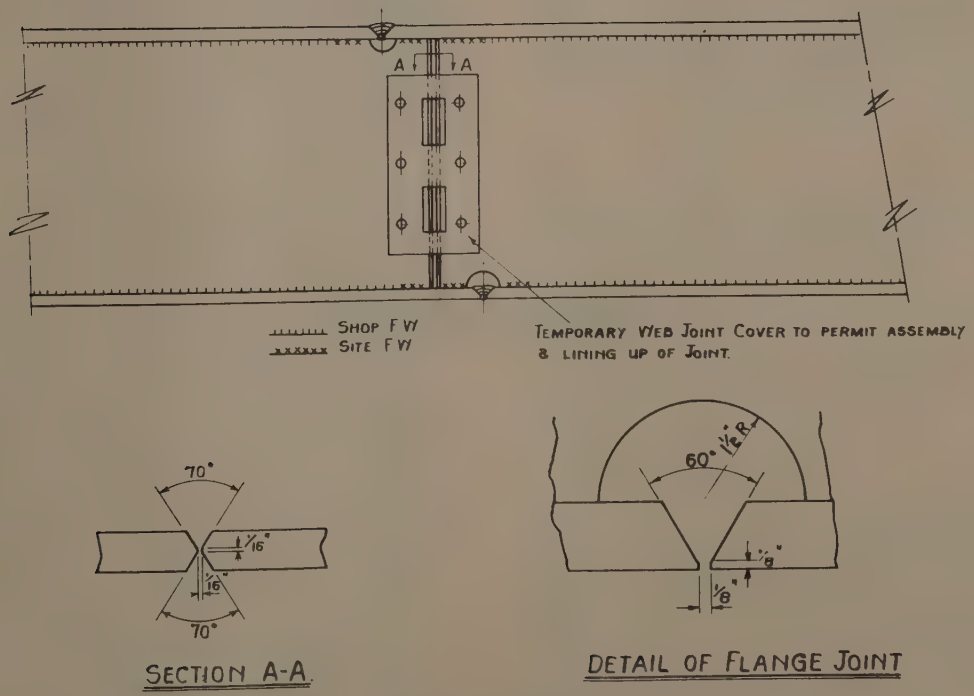


Fig. 17

In establishing a suitable procedure it is first necessary to consider the contractions which will take place due to the welding and arrange for the welding to proceed in such a manner that the final joint will be in as unrestrained a condition as possible.

The maximum contraction will be caused by the flange butt welds and these should be completed first. Depending on the thickness of the flange plates this contraction will be in the order of $\frac{1}{16}$ in. to $\frac{1}{8}$ in. and it is vital that the web joint shall have a root gap which is initially this much greater than that required. Thus if a $\frac{1}{16}$ in. gap is specified for the web joint it should actually be made with a $\frac{1}{8}$ in. to $\frac{3}{16}$ in. gap.

The individual runs of weld in the flange butt welds should be made starting from alternate edges, i.e. from left to right and then from right to left. This reduces the tendency for the joint to distort and is illustrated in Fig. 18.

The welding of the flange butt joints should be balanced to avoid residual stresses and possible cracking troubles with the second joint, i.e. for Joint 'A', deposit runs one and two then repeat for Joint 'B'. On Joint 'A' deposit runs three and four, then repeat for Joint 'B'. This procedure should be followed until the joints are completed. If the flanges are of large cross section two operators may weld simultaneously in balance.

When the welding grooves have been filled they should be back chipped or gouged to solid metal and a sealing run applied to each in the overhead position.

The flange butt welds should be completed by applying a light run deposited in the horizontal-vertical position on the edges of the plates. Alternatively, run on and run off plates may be used during welding, subsequently removed, and the ends of the welds dressed flush.

The web joint will now have moved such that its root gap is correct. It will be welded in the vertical position working vertically upwards. The second side should be chipped or gouged back to sound metal and the weld completed in the same manner.

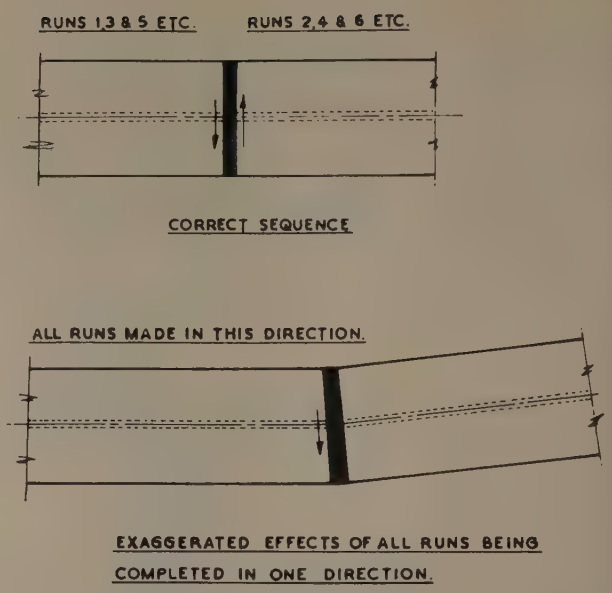

RUNS 1,3 & 5 ETC.
RUNS 2,4 & 6 ETC.
CORRECT SEQUENCE
ALL RUNS MADE IN THIS DIRECTION.
EXAGGERATED EFFECTS OF ALL RUNS BEING COMPLETED IN ONE DIRECTION.

Fig. 18

It is now only necessary for the web to flange fillet welds to be made. Those to the bottom flange will be in the horizontal-vertical position and those to the top flange in the overhead position.

The welders should be trained to follow the procedure outlined and there should not then be the necessity to specify this in detail for each joint, but the electrode sizes to be used and deposition rates should always be specified.

Acknowledgments

The writer wishes to express his appreciation to the Directors of Horseley Bridge and Thomas Piggott Ltd., Tipton, Staffs., for their permission to publish this paper.

Acknowledgments are made to the following for permission to publish illustrations.

Figure 3. B.S.C.A.

Figures 4, 5, 8, 10, 11, 12, 13, 14, 15. Quasi-Arc Ltd., Bilston.

Figure 6. R. J. Fowler Esq., B Sc.(Eng).

Volume Changes in Soils*

By E. LL. Morgan, A.M.I.Struct.E.

Introduction

The expression $\alpha p - s = u$ is used in a method of calculating the equilibrium moisture distribution in soils[1], where

α = a compressibility factor ;

s = the suction characteristic of the soil ;

u = the pore water pressure relative to the position of the water-table ; and

p = the pressure due to the overburden above the zone under consideration. The value p can also include any surface loading n due to a road, runway, or light structure, i.e.,

$p = h D_w + n$, where h = depth from the surface to the centre of the zone :

$$D_w = \frac{D_d(100 + m)}{100} = \text{bulk density of the soil,}$$

and D_d is the dry density : m = a value of moisture content assumed as a first approximation (this may be adjusted when the equilibrium moisture contents have been estimated once, by choosing the new value of m to give a better estimate of D_w).

The expression is applicable to all soils, since $\alpha = 0$ for incompressible soils and then $-s = u$; and in saturated clays $\alpha = 1$, with the equation becoming the more familiar $p - s = u$.

In unsaturated partially compressible soils intermediate to the limiting types, the overburden p is carried in part by the pore water and partly by the soil skeleton, the proportion depending upon the moisture state of the soil, being less on the pore water as the soil dries out. A change of volume occurs with change of moisture distribution in a compressible soil, but the intermediate group also exhibits some shrinkage on drying and swelling on wetting. It was suggested by Croney that the proportion of the overburden carried by the soil water, expressed in α, could be deduced from the shrinkage characteristics of the soil ; and so, in addition to soil moisture suction/moisture content relationships of local soils which were being obtained[2], it became necessary to take an interest in the volume/moisture content relationships ; and further, to devise if possible a single apparatus which would serve adequately both purposes over the practical range. (A typical soil moisture suction/moisture content relationship is illustrated in Fig. 1).

Interest in volume changes, due to causes other than application or relief of direct loads, has not attracted continued attention in Great Britain, but it is of some importance here as well as being a major problem overseas. With the current work leading in this direction, and because several post-graduate students

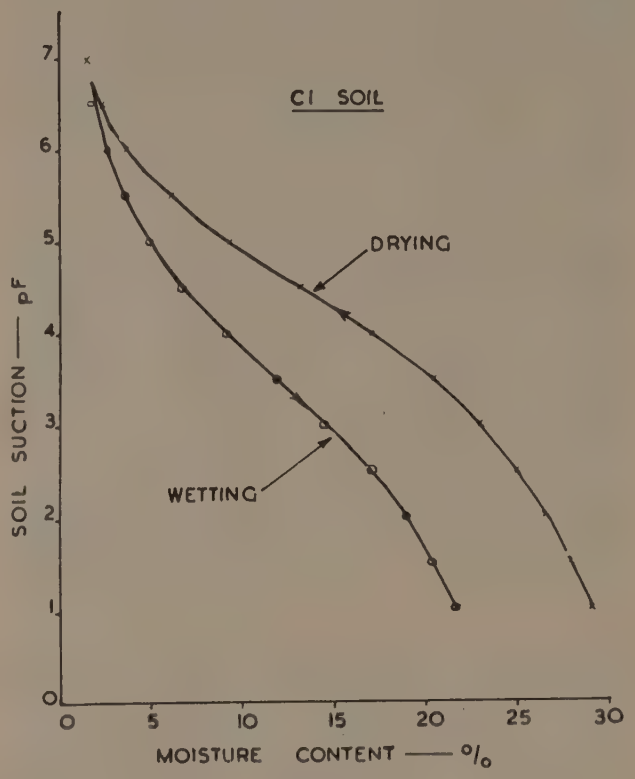


Fig. 1.

came from countries where vapour movements through and volume changes of soils were two of their main problems, it was decided to pursue the topic, notwithstanding that it is one of the less favoured fields of soil mechanics endeavour here.

An informal meeting at the 4th International Conference on Soil Mechanics and Foundation Engineering (London, 1957) offered considerable encouragement indeed, as it was made clear that the subject was by no means exhausted.

It is recognised that uneven support of house foundations and those of light structures is unlikely to be due to induced flow of soil, as the loads involved are small, probably less than one ton per square foot ; structural damage to such shallow founded buildings is due to differential movements associated with volume changes in clay-type soils with changes in moisture content. The major factors probably are :

i the potential causing moisture movements above the water-table ; and

ii the shrinkage and swelling characteristics of the soil mass itself.

In this paper, only a few of the many aspects are introduced.

* *Based on a lecture delivered to the Lancashire and Cheshire Branch of the Institution of Structural Engineers, at the College of Science and Technology, Manchester, on January 7th, 1958*

Causes of swelling and shrinkage

In most clays a certain degree of swelling takes place when the clay is relieved of part of its former load, for example, by excavation ; or of consolidation shrinkage when external loads are applied. In both these cases, the soil movements may be estimated using well-established methods based directly on or modifications of Terzaghi's theory of consolidation.

In the present context, the volume change results from the soil striving to reach suction equilibrium after the moisture state has been changed by climatic conditions, by the presence of actively growing vegetation, or by the actual construction covering the site, and appears to be associated more with soils which contain an appreciable percentage of influential clay minerals : building on such soils is almost certain to create the moisture change which leads to the difficulties.

The swelling of clay soils is due to the growth of adsorbed water layers on the surfaces of particles. The nature of such layers is considered to be more that of a solid than of a liquid, and their own adsorptive forces are very strong, which leads to the further growth of the layers during wetting opportunities, with an increase in the effective volume of each particle ; when the layers are in contact with each other, their continued growth creates an increase in the total volume of the soil structure. But some clay minerals have the property of adsorbing or releasing water from within their crystal structure with enhanced effects, and it is sometimes claimed that swelling and shrinkage are restricted to those soils which contain these specific clay minerals.

It is only since about 1924 that Ross[3] and others were able to show that the components of clay materials were essentially crystalline, and that there was a limited number of such crystalline components to which the name " clay minerals " should be applied. According to this concept, clays generally are thought to be composed of extremely small particles of one or more members of this small group of minerals ; these minerals are mainly hydrous aluminium silicates, with magnesium or iron wholly or in part replacing the aluminium in some minerals, and with alkalies or alkaline earths present as essential constituents in some others. Some clays may be of a dominant clay mineral, but most comprise a mixture. Some clay materials contain varying amounts of so-called nonclay minerals, such as quartz, calcite, feldspar, and there may be organic matter and water-soluble salts ; but the clay minerals are the components which determine the properties of the soils[4].

Grim[4] has advanced a tentative classification of clay minerals based on their structure and composition, and in which the descriptive terms " expanding lattice " and " non-expanding lattice " are applied to certain groups. The scheme is tentative because only of recent years have techniques (like X-ray analysis) so improved that more clear-cut classification should be possible ; and Grim recognises that his scheme may be modified in the future ; in fact, the succeeding page or so of his classical text criticises his own table, pointing out where alterations may be expected as work in this field progresses.

Another interesting table[5] shows that British clays, as far as present information leads one to conclude, are comprised mainly of illite and kaolinite, but the clay fraction of some deposits does include small quantities of montmorillonite, vermiculite, and halloysite clay minerals ; in particular it should be noted that small traces of montmorillonite and halloysite appear to influence *immensely* the physical properties of the soils in which they are present.

Illite, a group of minerals which has a non-expanding lattice, does not give rise to unusual properties, and the kaolinite mineral group also has a relatively stable structure.

Halloysite has a capacity for base exchange, but tends to neutralise or at least lessen the influence of any montmorillonite present.

The outstanding feature of the montmorillonite structure is that water and other polar molecules can enter between the individual cell layers, causing the lattice to expand and the mineral to swell, and accompanied by very great water adsorbing and holding capacity. Even small quantities present influence the properties of clays, so that they have high liquid limits, high natural moisture contents, and high volume change characteristics. There is also a very high base exchange capacity ; when sodium is the exchangeable base, water can enter more readily between the unit cell layers so that thicker adsorbed water layers are formed, and so sodium montmorillonite clays tend to swell very much more than the calcium or magnesium types. The reason for the solid character of these soils at relatively high moisture contents is associated with the orientation of the adsorbed water molecules and the consequent rigidity of the water in the first few molecular layers. Needless to state, drying out involves considerable shrinkage.

(The liquid limit for a Na-montmorillonite was found to be 731 per cent, and for a Ca-montmorillonite 285 per cent ; the plastic limit in each case was about 100 per cent).

Vermiculite has some influence due to its expanding lattice, but some doubt is expressed that the effects may be due to the presence of other clay minerals, and not specifically to the vermiculite.

It would appear that the order of volume changes in clay-type soils relative to the clay minerals present is :

greatest — montmorillonite,
intermediate — vermiculite and illite,
least — kaolinite ;

but also that, within the clay mineral groups, any replaceable bases present have profound effects in terms of volume change.

In the case of the clay at Chingford Reservoir, to cite a classic instance, there is about 70 per cent illite, 20 per cent kaolinite, and 10 per cent montmorillonite ; the 10 per cent has a distinct influence and accounts for the high shrinkage and high plastic properties.

To illustrate one of the many difficulties in this field, the lattice of expanding minerals can be completely collapsed by the entire removal of adsorbed water, especially so in the mineral montmorillonite ; and on subsequent wetting the minerals do not display even approximately their original properties. This function of the lattice may be responsible for the hysteresis effects in volume change/moisture content and soil moisture suction/moisture content curves when drying, wetting, re-drying and re-wetting curves are plotted. The extent of the modification to the properties on partial removal of adsorbed water is probably not easy to define, but would be useful to know.

Probably the surest way of defining clays liable to acute volume changes is by mineralogical analysis, but

this involves lengthy and different techniques not usual in soil mechanics laboratory practice ; and there are so few clay mineralogists and physical chemists directly interested in this class of work and its civil engineering outcome that it is unlikely that a direct tie-up with practical engineering problems will be evolved, at least not for a long time. It is this lack of a definite scientific measure that has prolonged the usefulness of other simpler methods of measurement.

The effects of vegetation, particularly fast growing trees, on shallow foundations, have been discussed by Ward[6, 7], whose publications have been particularly valuable in this connection ; Skempton[8] asks that more accurate case records should be published so that adequate solutions may be proposed.

The Measurement of Volume Variations

Conventionally the shrinkage limit is defined as the moisture content below which further loss of water by evaporation does not result in a further reduction of volume (Fig. 2). During some recent work on the

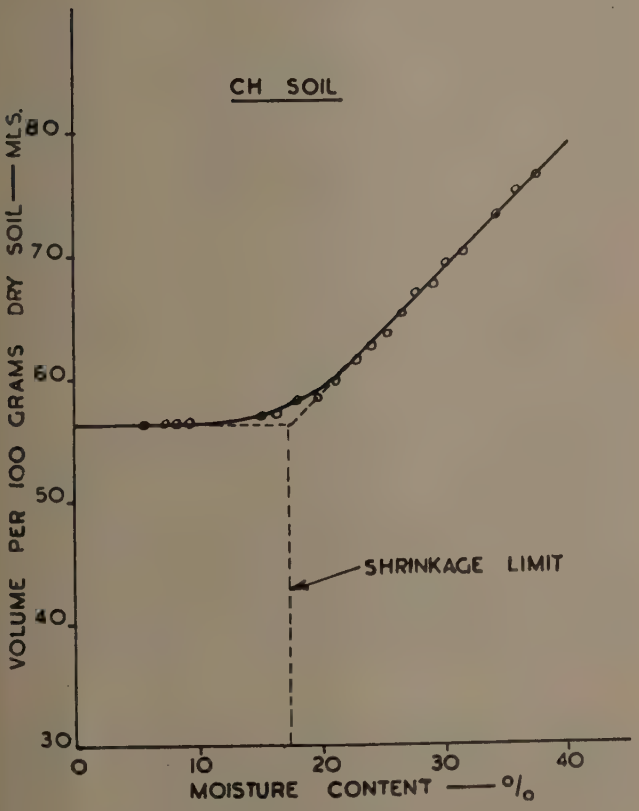


Fig. 2.

compressibility factor α, it was required that a reproducible test method be available for the determination of the shrinkage limit. It is surprising how many methods are in use in various parts of the world ; a comparative study was made of several of them, in some cases with slight modifications from the original descriptions as given in the literature. Those in common use include :—

(i) a linear observation method, after Tempany[9] ;

(ii) the bar linear shrinkage test, of Heidema[10] ;

(iii) a direct mercury displacement, or shrinkage limit method, using an apparatus similar to one originally designed by the Building Research Station (D.S.I.R.)[11] ;

(iv) a larger displacement apparatus, for specimens $1\frac{1}{2}$ in. or 2 in. dia.[11, 12], similar to one developed at the Road Research Station (D.S.I.R.) ;

(v) a mercury displacement, or magnification, method as used by Puri and his co-workers[13] ;

(vi) a direct displacement method, being a modification of that recommended by the American Society for Testing Materials[14, 15] ;

(vii) a gas displacement method, as originally described by B. N. Singh and P. B. Mathur[16] ;

(viii) an optical projection method ;

(ix) a consolidation method, using the oedometer.

Some of the methods are very unreliable, but a few give results reproducible to a good degree of accuracy with small specimens. The consolidation method was not pursued, as it can be argued that the mechanism causing volume change in this instance is not really parallel to the forces causing shrinkage as the term is normally interpreted, but it is a useful apparatus for giving an idea of the value of swelling pressures.

Of the many methods available, it appears fair to say that for normal engineering laboratory use the most suitable are :—

(a) the modified A.S.T.M.;

(b) the B.R.S. pattern ; and

(c) the Puri method.

The modified A.S.T.M. method overcomes the difficulties of spilling mercury into the collecting dish which the ordinary A.S.T.M. method allows. The test is carried out on a perforated perspex platform which rests horizontally in a glass funnel. The mercury displaced overflows into a small-bore graduated jar, the difference between the initial and final readings being the volume of the specimen. It is a very simple method and reasonably accurate.

A very useful mercury displacement apparatus, called for convenience the B.R.S. pattern, consists of three vertical cylinders, interconnected and partly filled with mercury. One cylinder has a plunger with a vernier attachment to determine the depth of plunger introduced into the cylinder, and so the volume of mercury displaced. Another cylinder has a needle point which can be adjusted to touch the mercury surface, at which instant a battery circuit is closed, and an electric light bulb comes on ; the third cylinder wider than the other two, has a well-fitting cap with a fixed plunger which immerses the specimen completely in the mercury. The difference between the vernier readings when the contact needle just touches the surface of the mercury with and without the soil sample, gives the corresponding volume of the specimen. The apparatus is used in conjunction with a series of steel cylinders of known volume, comparing the vernier reading given by the specimen with that given by a steel cylinder slightly larger than the specimen, the volume of the specimen being found by subtracting the difference in readings from the known volume. Less mercury is thus displaced, adding considerably to the accuracy of the apparatus[11]. (This pattern is on the market as standard equipment). A larger version[12] also gives very reasonable results.

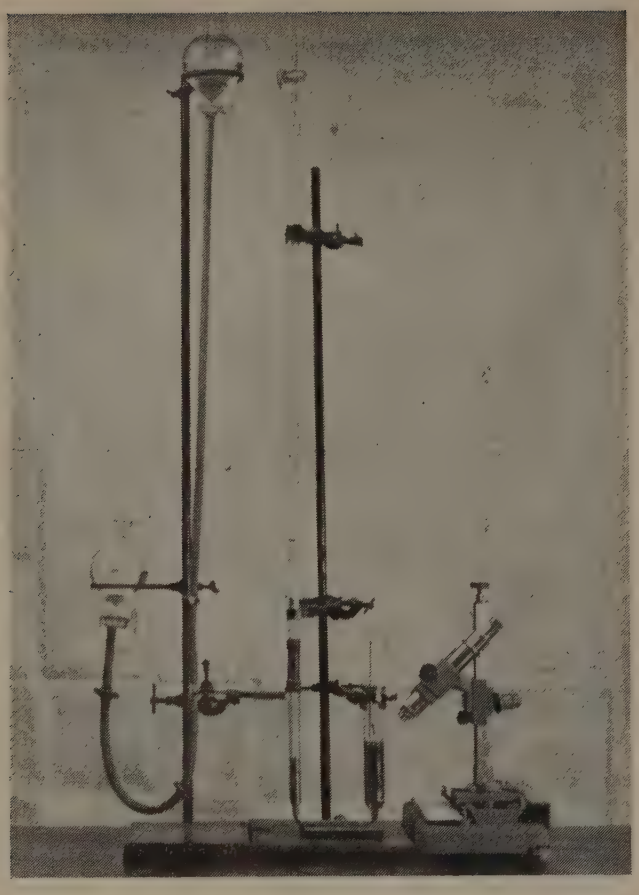


Fig. 3.

Puri and co-workers proposed the use of smaller samples ; and found the volume of the sample using a simple magnification technique (Fig. 3). The apparatus is based on the principle of balancing columns, and depends on the large difference of densities of liquids used in two U-tube limbs, and also the difference in diameters of the limbs, to produce the required magnification ; the liquids used are water and mercury. The apparatus, like most displacement methods, may be criticised on several counts ; air bubbles become trapped in the mercury ; specimens deform under the pressure of the mercury ; and mercury may cling to the specimens and even enter them when shrinkage cracks occur at the drier stages. Frequent cleaning both of the apparatus and the mercury is called for in most of these methods, and generally eliminates the first stated difficulty, but the third is sometimes quite serious. However, the Puri magnification method reduces error in the actual measurement of the volume of a specimen to the order of 0.001 ml ; and if the operator exercises care, he will find it a sensitive and useful apparatus. A further slight modification will probably result in an exceptionally accurate volume measuring device.

Heave and settlement of buildings

The purpose of the tests is to define the shrinkage limit, after which it is possible to obtain a tentative indication of the possible shrinkage movements at the surface. Of the many interpretations, that of Ward[6] is well-known, in which he introduced a ratio indicative of the potential shrinkage capacity of a clay. The shrinkage capacity is defined as the decrease in volume, expressed as a percentage of the volume at the shrinkage limit, when the moisture content is reduced from the maximum value at which it exists naturally to its shrinkage limit. This gives a measure of the possible shrinkage movements at the surface of a clay deposit for a given climate and vegetation.

The A.S.T.M. shrinkage ratio is similar in intent, being " the ratio between a given volume change, expressed as a percentage of the dry volume, and the corresponding change in moisture content above the shrinkage limit, expressed as a percentage of the weight of the oven-dried soil ".[14]

These definitions were introduced as indicators rather than as specific means of measurement. In many parts of the world (Israel, Australia, South Africa, to name a few) the movements, due to swelling of the soil after rain and then re-shrinkage, are so large that some other means of estimating heave and settlement quantitatively are essential, and broad indicators are not enough.

Again, the shrinkage limit appears to be higher for the same soil tested from a moisture condition somewhere near the liquid limit than from a drier condition, and also different if a wetting method is used. It has always been recognised that, from greater water contents, clays shrink considerably due to the amount of water removed, whilst from the lower range of water contents there may be practically no further shrinkage ; but it is assumed that the shrinkage limit is the same in each case, which is not necessarily so. Similarly, undisturbed and remoulded specimens give differing values ; with some soils the difference may be only 2-3 per cent, but with others it may be much greater. As in other branches of soil mechanics, the test should be parallel to the specific field circumstances ; in the present context, this suggests that the moisture history of the site immediately prior to construction should be known, and a shrinkage value adopted from the appropriate drying or wetting test.

A better idea of possible soil movements may be obtained from the use of soil moisture suction/moisture content curves in conjunction with volume change/moisture content curves, rather than of shrinkage curves on their own ; and if the results obtained still need liberal interpretation, they will probably be more accurate than those by other means. In other words, attempt a prediction of the change in moisture content that will occur due to a site being covered by a structure and use this along with a specific determination of shrinkage limit, for which the whole curve is available.

Samples of soil from boreholes should be taken at equal intervals of depth prior to construction so that the initial moisture content distribution of the soil profile is known. It is now necessary to find the likely final equilibrium moisture condition attained by the soil after the structure has been completed. The estimate is based on the equation $\alpha p - s = u$; as α, p, and u can be determined, then s, the suction, can be calculated, and converted to the pF scale, after which the corresponding equilibrium moisture content for the point under consideration can be read off the appropriate suction/moisture content curve for the soil type encountered in the soil profile. For most clays the value of α can probably be taken as unity without involving too great an error ; a sandy clay might have a value as low as 0.5, but 0.9 or 1.0 may be adopted for *C H* soils. (Methods are available for the estimation of α, but it is not the intention to discuss them in this lecture).

Heave or settlement estimates of a structure on a susceptible soil then involve knowledge of these items :

(a) The proposed surface load intensity ;

(b) The final equilibrium moisture condition ;

(c) The volume change of the soil profile associated with the change from the initial moisture content to the equilibrium value, i.e. the appropriate volume-change test results.

These can be reasonably assessed, and the total volume change integrated over the equal intervals of depth within the soil mass likely to be affected, usually the height above the water table unless this is very deep.

Other factors must be remembered. The uniformity or otherwise of the soil types and the thickness of the layers ; edge effects, i.e. moisture movements into or from the soil adjacent to the structure ; effects of large trees near the structure, can all modify the result, but cannot be allowed for directly in any estimate.

The influence of edge effects, that is, the lateral movement of moisture, can be minimised by constructional measures, usually an impervious apron at least 5 ft. wide around the structure, assuming the adjacent areas are grass-covered ; if the adjacent areas are bare, then the apron would need to be much wider. But there may be an added danger if tree roots are covered, and hence deprived of their direct source of moisture. Careful positioning of a structure relative to existing trees is important, and new trees should be sited such that the transpiration influence can be considered negligible. If an old tree has been cut down, the building should not be sited on or near this position, as the soil previously exploited by the tree will tend to swell as it regains moisture.

If the surface load intensity is large, then the problem becomes one of settlement due to consolidation, effectively masking the effects being discussed.

To estimate the worst possible heave likely to occur, a knowledge of the drought water table level, perhaps estimated from rainfall data, and of the mean position of the water table are required ; and the equilibrium moisture condition for both cases determined for use in the estimate. It is essential that the appropriate suction curve (wetting or drying as the case dictates) be used depending on whether the likelihood is of swelling or shrinkage. The difference between the estimated drought moisture contents and those for the mean position are used to calculate the volume change at equal intervals of depth.

A low water-table is often stated as one of the requisite conditions for very large heaving or shrinkage. The greater the depth of the water-table the greater will be the soil mass over which the movement is integrated ; but it is also true that considerable movements have occurred on sites where the water-table has been at a depth of only a few feet.

Shrinking and Swelling Pressures

During the process of expansion, that is, during the rainy season, colloidal clays and expansive soils generally develop high pressures, particularly if the soil is confined. Dawson[17] measured swelling pressures as high as 30,000 lb. (13-14 tons) per sq. ft. ; but the resulting swelling pressures were much reduced if some slight expansion were permitted before the restraint came into action. The pressures were measured in the laboratory using standard oedometers ; undisturbed specimens from the site were placed in the cells, loads equivalent to the overburden pressure applied, and then the specimens were flooded and allowed to swell. The loads which had to be applied to the specimens to regain their original thicknesses were considered the corresponding swelling pressures.

Swelling pressures of 10 kg/sq. cm. (about 10 tons per sq. ft.) were also reported by Goldberg and Klein, 1952.[18]

Apart from moisture changes, the initial soil density and particularly its natural structure are also important factors influencing volume change. The objectionable features found in the oedometer approach are that, firstly, the structural arrangement of the soil particles is sometimes altered during the preparation of the specimen ; but of greater importance is the fact that, in nature, the moisture variations are often in a range below that of full saturation ; that is, the field condition of the soil is not necessarily reproduced in earlier methods used for measurement of swelling pressures.

Alpan's apparatus[19] is a modification of the continuous method of suction measurement[20], and overcomes these difficulties. The sintered glass suction plate fits into a special cell which holds the undisturbed specimen, whose suction value can be varied. A metal plate rests on the soil, and transmits the swelling pressure through a rod to a proving ring. No extravagant claims have been made about the efficiency of this apparatus, other than that it appears to be a more logical approach ; but tentative experiments along similar lines (in the Municipal Engineering Department, The Manchester College of Science and Technology) suggest that further development and use of the apparatus are warranted.

It does not seem possible yet to obtain a clearly defined correlation between swelling pressures and volume expansion, although these have been measured at different moisture contents and dry densities.

It is essential for any design method that test measurements of swelling pressures and of free swelling under constant load should be available. This was unequivocally stated by Salas and Serratosa, Spain,[21], who studied design problems in connection with swelling clays, and on the basis of such tests predicted approximately the movements of light foundations from curves of swelling pressure against allowed swelling. They also discussed different constructive measures, and indicated an empirical method for the calculation of reinforcement in footings constructed on expansive soils. Their work is not entirely conclusive, but is a notable advance in this field, and supports the contention that a reliable apparatus for measurement of these forces is still required.

A new device for measuring shrinkage pressures was put forward by Palit and Joshi[22], who obtained values ranging from 0.3 to 0.4 tons per sq. ft. Their important conclusion, however, was that most of the force generated inside a soil by shrinkage forces is used up in compacting the soil, and only a small amount is available for exerting pressure on an adjacent structure ; even this small amount will not act on a structure when the soil cracks or does not shrink towards the structure, that is, they confirmed that shrinkage pressure is not such a damaging factor to underground structures as swelling pressure.

Other means of definition

For British conditions, an examination for possible volume changes on the basis of estimation of moisture

changes, outlined earlier, may be considered too exacting a procedure ; and broader indications can suffice. It is well-known that certain structural methods can cope with the difficulties, and soil classification with these in mind may be all that is necessary. Abroad, too, certain points will be sought before indulging in an elegant laboratory examination.

The following indicators are generally associated with the likelihood of large heave or settlement.

(i) The presence of a heavy clay, that is, a soil with a high percentage of clay-size particles ; the extended Casagrande classification scheme, however, includes the intermediate clays also, i.e. CH, OH, MH, OI and CI groups, as all exhibiting volume-change tendencies. In broad terms a high liquid limit may imply a difficult soil. Heidema[10] claimed that his bar-linear shrinkage test and the Atterberg limits give quite definite relationships. The bar-linear shrinkage is expressed as percentage of original length, which can be converted into terms of volumetric shrinkage per cent of original volume ; he claimed also that the bar-linear shrinkage test is a better identifier of soil types than the plasticity index, but, without supporting this argument, there is no doubt that he and others have established that high liquid limits and plasticity indices are almost always associated with soils which give large volume changes.

(ii) The activity of soil ; as defined by Skempton[5], this is the ratio, activity $= \frac{plasticity\ index}{percentage\ clay\ fraction}$, which for any particular clay is approximately constant. For two clays with the same value of clay fraction content but different plasticity indices, it is stated that the one with the higher plasticity index is more colloidally active than the other with the lower P.I. Clays are broadly classified as being

inactive when the activity < 0.75
normal ,, ,, ,, is 0.75 to 1.25, and
active ,, ,, ,, > 1.25

Simply, the activity is related to the type of clay minerals present, the relationship having been established by collecting together published information to which Skempton has added further results. It is a more valuable indicator than the plasticity index for classification purposes, and only needs two tests that are usually carried out in most laboratories as normal routine. Values from 0.90 to 1.0 and above would suggest that fuller investigation into the volume-change properties of the soil would be warranted.

(iii) The degree of desiccation of the soil on site is of significance in tropical and semi-arid regions, where high temperatures cause extreme drying. The " degree of desiccation " is not clearly defined, and any description is based on visual inspection of the soil profile in a freshly excavated pit, a shattered profile or well-developed slickensides being evidence that the soil has been dried to the extreme low moisture content of desiccation. In the dry season this can extend to depths of 20 feet or more, with open shrinkage cracks to about the same depth. The rainy season will cause a loss in soil strength due to the softening effect of water in the cracks, and hence an initial settlement, but the ultimate heave is very large and damaging to buildings as is frequently demonstrated in Israel, South Africa and other regions.

(iv) A deep-lying water table is contributory, but on its own is not sufficient evidence. Jennings[23], South Africa, stresses that material heaving only occurs when the following exist concurrently :

(a) a clay profile of high activity ;

(b) a low water table, a greater heave being experienced since the swelling is integrated over a greater zone which may extend from 15 to 100 feet ; and

(c) a desiccated condition.

(v) Wide variations in rainfall intensity between the seasons and high temperatures promote desiccation ; water table levels vary considerably, and edge effects result from the extremes of heavy to low or nil rainfall ; (iii), (iv) and (v) are usually considered together as indications.

(vi) Shrinkage rate, or shrinkage factor, as a definition of the volume-change properties of a soil, was introduced by Haefeli and Amberg[24]. They recognised that the shrinkage limit of a sample prepared at a higher initial moisture content is greater than that of the same soil prepared at a lower moisture content ; but observed that all shrinkage lines in the first phase of the shrinking process, when produced back, met at a common point called the shrinkage centre, irrespective of the initial moisture content (Fig. 4). The gradients of the shrinkage lines vary because of the different unit dry weights of the prepared specimens. The shrinkage centre occurs at a distance from the vertical axis of the plot, and this distance is termed the shrinkage rate or factor ; it is a constant value for a particular clay. Some independent tests[15] indicated that the difference in values obtained with different clay-type soils was distinct but not very large, probably due to the narrow range of

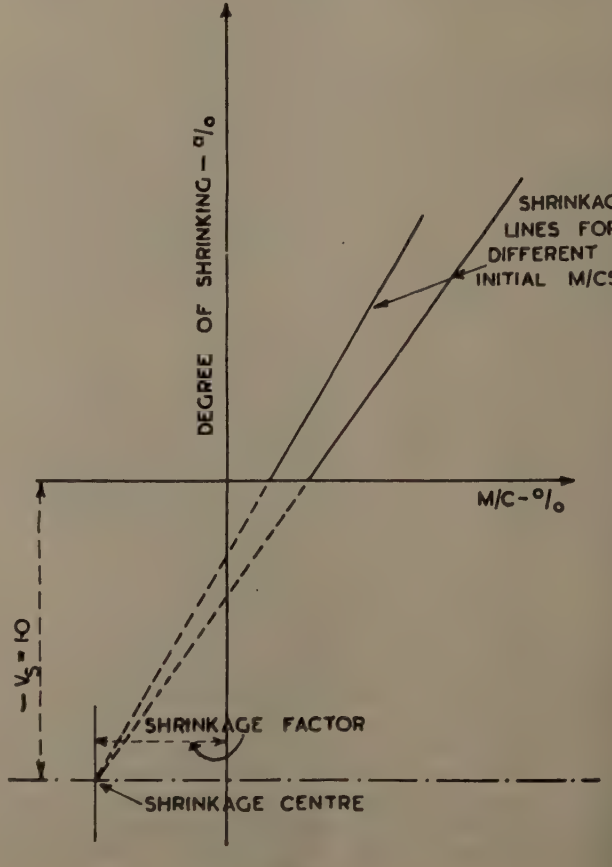


Fig. 4.

soils that were tested; but even so, equally clear distinctions could be observed by other means. There has not been much work done along these lines, and so the only immediate conclusion is that the shrinkage factor is a possible indicator of volume-change characteristics, but it is too early to claim it as a clear-cut classification.

Shrinkage capacity (Ward), and shrinkage ratio (A.S.T.M.) are both valuable indicators, which have been defined earlier.

General Remarks

The change in volume of a soil due to variations in moisture content plays an important part in soil studies in many countries.

The need for direct measurement of movements in the field is emphasised by Baracos and Bozozuk[25] of the National Research Council of Canada. Whereas in South Africa the problems are caused mainly by swelling, and in Great Britain more often by shrinking, in Canada both problems exist. Detailed studies are being carried out both in Winnipeg and Ottawa, where ground movements are being measured to depths of 8 feet and more, using multirod gauges and concentric telescopic ground movement gauges. The problems were brought to their notice mainly because of the trend towards houses without basements. Their work is interesting because it accepts the suction, or pressure deficiency, theory as a fundamental approach. For broad identification purposes high plasticity indices, liquid limits, and activity ratios are the accepted indicators. Differential settlements of brick dwellings of 3 to 4 inches have been observed in Ottawa, and occasionally one foot, but as great as this is usually associated with the presence of trees. The susceptibility of poorly compacted backfill was particularly noted.

Zeitlen and others[26] used local clays for embankments in Israel because of the costly transport and quarry charges for other materials. Because of the swelling nature of the clays there was a rise, rather than a settlement, of those embankments used in water supply practice. The highly plastic clays contained a large content of Ca-montmorillonite.

It does appear that a large amount of empirical information exists about this problem, but that there is still room for further closer investigations of a more fundamental nature. While not perhaps of major importance in British conditions, it is obviously of widespread interest throughout the world, and it is unfortunate that some studies are repeating work done elsewhere. The need for a clearing house of information is obvious in this as in other spheres of technology, and it is to be hoped that some central body might encourage an organisation to cope completely and finally with the problem, thus saving the recurring expenditure on remedial measures.

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Institution Notices and Proceedings

PRESIDENTIAL ADDRESS
Session 1958-59

A General Meeting of the Institution of Structural Engineers will be held at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 2nd October, 1958, at 6 p.m., when Mr. G. S. McDonald, M.I.Struct.E., M.I.C.E., M.I.Mun.E., will be installed as President for the Session 1958-59 and will give the Presidential Address.

PRESIDENT'S ENGAGEMENTS—OCTOBER

Monday, 6th October, 1958
The President will attend the opening of the Exhibition which is being held in connection with the Fiftieth Anniversary Conference of the Institution.

Tuesday, 7th October to Friday 10th October, 1958
Fiftieth Anniversary Conference.

Monday, 13th October, 1958
Scottish Branch Opening Meeting, Glasgow.

Tuesday, 14th October, 1958
Scottish Branch Annual Dinner, Glasgow.

Wednesday, 15th October, 1958
Yorkshire Branch Opening Meeting, Leeds.

Friday, 17th October, 1958
Western Counties Branch Opening Meeting, Bristol.

Saturday, 18th October, 1958
Midland Counties Branch Annual Dinner, Birmingham.

Tuesday, 21st October, 1958
Northern Counties Branch Opening Meeting, Middlesbrough.

Thursday, 23rd October, 1958
Ordinary Meeting in London.

Friday, 24th October, 1958
Midland Counties Branch Opening Meeting, Birmingham.

Tuesday, 28th October to Friday, 31st October, 1958
Lancashire and Cheshire Branch Jubilee Celebrations, Manchester.

FIFTIETH ANNIVERSARY CONFERENCE

The Fiftieth Anniversary Conference will be held in London on 7th to 10th October, 1958. It will commence with a reception at Londonderry House, Park Lane, by the President, Mr. G. S. McDonald. The Technical Sessions will be held on the mornings of Wednesday, Thursday and Friday, 8th to 10th October, and the Conference will terminate with a Banquet and Dance at the Dorchester, Park Lane, on Friday, 10th October. Alternative tours and visits of general interest have been arranged and the programme includes a variety of social functions and entertainments.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, 23rd October, 1958

Ordinary General Meeting for the election of members 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when the following paper will be given by Mr. L. E. Ward, M.I.Struct.E., " Aluminium and Tubular Steel as applied to the Construction of some recent Aircraft Hangars ".

Thursday, 13th November, 1958

Ordinary Meeting at 6 p.m., when Mr. D. A. G. Reid, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E. (Member of Council), will give a paper on " Technical Education and Professional Training ".

Thursday, 27th November, 1958

Ordinary General Meeting for the election of members 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when a paper on " Modern Methods of Fabricating Structural Steelwork " will be given by Mr. M. F. Palmer, M.I.Struct.E., M.I.C.E., and Mr. R. J. Fowler, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., A.M.I.Mech.E.

Thursday, 11th December, 1958

Ordinary Meeting at 6 p.m., when a paper will be given by Mr. S. M. Reisser, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., Mr. D. Bolton, B.Sc.(Eng.), and Mr. K. M. Wright, B.Sc.(Eng.), A.M.I.C.E., on " The Design and Construction of Pelham Bridge, Lincoln ".

Thursday, 18th December, 1958

Ordinary General Meeting 5 p.m. This meeting is for the election of members and is open only to corporate members of the Institution.

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

SPECIAL JUBILEE ISSUE OF " THE STRUCTURAL ENGINEER "

Copies of the Special Jubilee Issue of " The Structural Engineer " which was published on the 21st July to mark the Fiftieth Anniversary of the Institution are available, price 10s. 6d. (by post 12/-), on application to the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1.

EXAMINATIONS, JANUARY, 1959

The Examinations of the Institution will be held in the United Kingdom and overseas on Tuesday and Wednesday, January 6th and 7th, 1959 (Graduateship), and Thursday and Friday, January 8th and 9th, 1959 (Associate-Membership).

THE INSTITUTION OF STRUCTURAL ENGINEERS APPOINTMENT OF DEPUTY SECRETARY

Applications are invited for the appointment of Deputy Secretary of the Institution of Structural Engineers, with a view to eventual appointment as Secretary.

Applicants should preferably be corporate members of a Chartered Engineering Institution, have appropriate administrative experience and be between the age of 35 and 50.

The commencing salary will be from £1,800 to £2,000 per annum according to experience.

The closing date for the receipt of applications is the 24th October, 1958.

For further details apply to the Secretary of the Institution, 11, Upper Belgrave Street, London, S.W.1., the envelope to be marked " Deputy Secretary ".

LONDON GRADUATES' AND STUDENTS' SECTION

The opening meeting of the Session will be held at 11, Upper Belgrave Street, London, S.W.1., on Tuesday, 14th October, 1958, when Mr. Johnson Marshall, A.R.I.B.A., A.M.T.P.I., will give a talk on " Design of Cities ".

On Tuesday, 4th November, 1958, an " Engineers' Forum " has been arranged. The speakers, will be : Mr. D. N. Mitchell, M.I.Struct.E., M.I.C.E., Mr. M. E. R. Little, M.A., A.M.I.Struct.E., A.M.I.C.E., and Mr. J. E. C. Wehrle, A.M.I.Struct.E.

On Monday, 1st December, 1958, a meeting will be held when films of engineering interest will be shown.

Hon. Secretary : R. M. Amodia, B.E., 21, Wetherby Gardens, London, S.W.5.

EXAMINATIONS

PREPARATION FOR THE EXAMINATIONS OF THE INSTITUTION BY ATTENDANCE AT TECHNICAL COLLEGES

A Candidate for Graduateship or Associate-Membership may be able to attend a Technical College ; these notes are intended to guide him in choosing the most suitable instruction.

PREPARATION FOR THE GRADUATESHIP EXAMINATION

Technical Colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix 11, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At Technical Colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

PREPARATION FOR THE ASSOCIATE-MEMBERSHIP EXAMINATION

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering. These cover only part of the requirements for the Associate-Membership Examination.

Colleges in List ' A ' provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

List ' A '

Bath Technical College.
Belfast College of Technology.
Birmingham College of Advanced Technology.
Bolton Technical College.
Bradford Technical College.
Bridgend Technical College.
Bristol College of Technology.
Chesterfield College of Technology.
Coatbridge Technical College, Lanarkshire.
Derby and District College of Technology.
Dudley and Staffordshire Technical College.
Glasgow Royal College of Science and Technology.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith College of Art and Building.
Manchester College of Technology.
Middlesbrough, Constantine Technical College.
Nottingham and District Technical College.
Portsmouth College of Technology.
Salford, Royal Technical College.
South-East London Technical College, Worseley Bridge Road, S.E.26.
South-West Essex Technical College, Walthamstow, E.17.
Southampton Technical College.
Stafford County Technical College.
Stockport College of Further Education.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in List " B " provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete.

List " B "

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea Polytechnic, S.W.11.
London, Northampton College of Advanced Technology, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Newcastle upon Tyne, Rutherford College of Technology.
Plymouth and Devonport Technical College.
Preston, Harris Institute.
Rotherham College of Technology.
South-East Essex Technical College, Dagenham.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.
West Ham College of Technology.

Students are advised to take the organised courses in Structural Engineering where these are available.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The Branch will celebrate the Fiftieth Anniversary of the Institution from 27th October to 1st November,

1958, when the following arrangements have been made :

Tuesday, 28th October, 1958

The inaugural meeting of the Session will be held in Manchester, commencing at 7.15 p.m., at which the Chairman's Address will be given by Dr. D. D. Matthews, M.I.Struct.E., M.I.C.E., A.M.I.Mech.E. The address will be preceded by a Hot Pot Supper and may be followed by a suitable film. A coach will run from Liverpool.

Wednesday, 29th October, 1958

A SYMPOSIUM ON STRUCTURAL ENGINEERING will be held when papers will be presented by Professor Hardy Cross, Sir Richard Southwell and Professor Sir Alfred Pugsley, with discussion by Professors J. A. L. Matheson, W. Merchant and J. B. Owen.

The Symposium will take place at the College of Science and Technology and at the University, Manchester. During the proceedings Professor Cross will be presented with the Gold Medal of the Institution in recognition of his contributions to structural engineering. Admission is limited to ticket holders (see below).

CIVIC RECEPTION. The Lord Mayor and Corporation of the City of Manchester graciously invite members of the Branch to a Civic Reception in the Town Hall. Admission by invitation only (see below).

Friday, 31st October, 1958

The Jubilee Banquet will be held at the Midland Hotel, Manchester.

. . . . .

An Exhibition entitled " The Diversity of Structural Engineering " in the Exhibition Hall, College of Science and Technology, Manchester, will be open from 10.30 a.m. to 7.30 p.m. Monday 27th October, to Friday, 31st October, and from 10.30 a.m. to noon on Saturday, 1st November, 1958. Films will be shown during the morning, afternoon and evening.

Inaugural Meeting : tickets (5s.) from H. J. Dowling, A.M.I.Struct.E., 82, Station Road, Hadfield, nr. Manchester.

Symposium : application for registration (5s. including buffet lunch and transport between sessions) to M. D. Woods, A.M.I.Struct.E., 8, Dennison Road, Cheadle Hulme, Cheshire.

Civic Reception : applications for limited invitations (from Branch members only) may be made to J. L. Robinson, A.M.I.Struct.E., Department of Structural Engineering, College of Science and Technology, Manchester 1.

Jubilee Banquet : tickets (2 gns. each) are available from Mr. J. L. Robinson at the above address.

The following meetings have been arranged for November and December :

Tuesday, 11th November, 1958

Dr. C. T. G. Boucher, A.M.I.Struct.E., A.R.I.B.A., on " A History of Structural Engineering in Lancashire and Cheshire ".

Wednesday, 3rd December, 1958

Joint Meeting with the Reinforced Concrete Association to hear a paper by Mr. W. S. Watts, A.M.I.Struct.E., A.M.I.C.E., on " Points of Structural Interest at Calder Hall ".

Unless otherwise stated meetings will be held at the College of Science and Technology, Manchester, commencing at 6.30 p.m., and will be preceded by refreshments.

MERSEYSIDE PANEL

The following meeting has been arranged :

Wednesday, 19th November, 1958

Mr. N. T. Barrett, B.Sc.(Eng.)., A.M.I.Struct.E., on " Housing the Dounreay Fast Reactor ", at the College of Building, Clarence Street, Liverpool, commencing at 6.30 p.m., preceded by light refreshments from 5.30 p.m.

Joint Hon. Secretaries : J. L. Robinson, A.M.I.Struct.E., 314, Northenden Road, Sale, Manchester ; M. D. Woods, A.M.I.Struct.E., 8, Dennison Road, Cheadle Hulme, Cheshire.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :

Saturday, 18th October, 1958

Annual Dinner and Ladies' Evening at the Botanical Gardens, Birmingham.

Friday, 24th October, 1958

The Chairman's Address will be given by Mr. B. C. Britton, M.I.Struct.E., F.R.I.C.S.

Wednesday, 12th November, 1958

Dr. F. W. Gifford, M.I.Struct.E., A.M.I.C.E., on " Factory Production of Precast Concrete ", at the Electricity Demonstration Hall, Irongate, Derby, at 6.15 p.m., preceded by tea at 5.30 p.m.

Tuesday, 25th November, 1958

Joint Meeting with the Nottingham, Lincolnshire and Derbyshire Architectural Association at 64, St. James' Street, Nottingham, 7. p.m.

Friday, 28th November, 1958

Joint Meeting with the Birmingham and Five Counties Architectural Association.

Mr. A. G. Ling, B.A., F.R.I.B.A., M.T.P.I., on " International Union of Architects 1958 Congress in Moscow ".

Unless otherwise stated, all meetings will be held at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6.30 p.m. Tea will be served from 5.45 p.m.

Hon. Secretary : John R. Chaffer, M.I.Struct.E., 107, Jockey Road, Sutton Coldfield, Warwickshire.

GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :

Friday, 31st October, 1958

Mr. Ove Arup, C.B.E., M.I.Struct.E., M.I.C.E., on " Future Trends in Structural Engineering ".

Friday, 7th November, 1958

Dr. L. G. Booth on " Advanced Ideas in Timber Structures ".

Friday, 5th December, 1958

Mr. H. C. Husband, B.Eng., M.I.Struct.E., M.I.C.E., M.I.Mech.E. (Honorary Editor) will introduce a sound film of the history and construction of " The Jodrell Bank Radio Telescope ".

Meetings will be held at the Birmingham Exchange and Engineering Centre, Stephenson Place, Birmingham and will start at 6.30 p.m. Tea will be served from 6 p.m. and visitors will be welcome.

Hon. Secretary: F. A. Butterworth, "Roscrea," Tansey Green, Pensnett, Brierley Hill, Staffs.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged:

Wednesday, 1st October, 1958

At Newcastle. An address will be given by Mr. D. W. Portus, M.I.Struct.E. (Chairman-elect).

Tuesday, 21st October, 1958

At. Middlesbrough. Installation of Chairman. Mr. D. W. Portus, M.I.Struct.E., will give the Chairman's Address.

Tuesday, 28th October, 1958

Joint Meeting with the Northern Architectural Association.

At Middlesbrough. Professor W. Fisher Cassie, Ph.D., M.S., F.R.S.E., M.I.Struct.E., M.I.C.E., on "Settlement Problems in Mexico City".

Wednesday, 5th November, 1958

At Newcastle. Lecture to be arranged.

Tuesday, 2nd December, 1958

At Middlesbrough. Mr. D. R. Sharp, M.B.E., B.Sc., A.M.I.Struct.E., A.M.I.C.E., on "Concrete Roads—Some Recent Developments".

Wednesday, 3rd December, 1958

The above paper will be repeated at Newcastle.

Meetings in Middlesbrough will be held in the Cleveland Scientific and Technical Institution, and meetings in Newcastle will be held in the Neville Hall.

All meetings will commence at 6.30 p.m., and will be preceded by buffet tea at 6 p.m.

Hon. Secretary: H. W. Dowe, A.M.I.Struct.E., 2, The Crescent, Saltburn-by-the-Sea, Yorks.

NORTHERN IRELAND BRANCH

The following meetings have been arranged:

Tuesday, 11th November, 1958

Mr. A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., A.M.I.C.E., will be installed as Chairman and will give an address entitled "The Chairman presents . . . .".

Tuesday, 2nd December, 1958

Mr. J. McClure, B.Sc., A.M.I.Struct.E., A.M.I.C.E., will report on The Third International Congress on Prestressing, Berlin, May, 1958.

This meeting will be attended by the members of the Northern Ireland Association of the Institution of Civil Engineers.

Meetings will be held at the College of Technology, Belfast, commencing at 6.45 p.m.

Hon. Secretary: L. Clements, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., 3, Kingswood Park, Cherryvalley, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged:

Monday, 13th October, 1958

Mr. W. Heigh, M.I.Struct.E., will give the Chairman's Address.

Tuesday, 14th October, 1958

Annual Dinner and Dance at the Grosvenor Restaurant, Gordon Street, Glasgow, 6.30 p.m.

Tuesday, 4th November, 1958

Mr. C. M. Wilson, M.I.Struct.E., M.I.C.E., on "Structural Steel Framework and Overhead Gantries at Leith Fertiliser Works for S.A.I. Ltd.", at the Adelphi Hotel, Coburn Street, Edinburgh, 6.15 p.m.

Tuesday, 25th November, 1958

Joint Meeting with West of Scotland Branch, The Institute of Welding.

Mr. David Watson on "Application of Welding to Structural Engineering".

Tuesday, 9th December, 1958

Mr. K. F. Geesin, M.I.C.E., on "Structural Aspects of Ravenscraig Melting Shop".

Unless otherwise stated meetings will be held at The Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, commencing at 7 p.m.

Hon. Secretary: W. G. Cantlay, B.Sc., A.M.I.Struct.E., A.M.I.C.E., 3, Blairbeth Terrace, Burnside, Glasgow.

SOUTH WESTERN COUNTIES BRANCH

The following meetings have been arranged:

Thursday, 6th November, 1958

The Chairman's Address will be given by Mr. E. W. Howells, M.I.Struct.E., at the Grand Hotel, Plymouth, 7 p.m.

Friday, 5th December, 1958

Mr. Walton Lund on "The Construction of the new Queen Elizabeth Dock at Falmouth".

In Truro, time and venue to be notified.

Hon. Secretary: C. J. Woodrow, J.P., "Elstow", Hartley Park Villas, Mannamead, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged:

Wednesday, 5th November, 1958

At Cardiff. Mr. Wallace A. Evans, M.I.Struct.E., will give the Chairman's Address.

Monday, 10th November, 1958

The Chairman's Address will be repeated at Swansea.

Saturday, 15th November, 1958

The Chairman's Address will be repeated at Colwyn Bay.

Friday, 12th December, 1958

Joint Meeting with the Institution of Civil Engineers and Institution of Municipal Engineers.

At Cardiff. Professor B. Neal, M.A., Ph.D., A.M.I.C.E., Dr. A. Harvey and Lt.-Col. A. Borlase, T.D.M.I.C.E. will present a Symposium of papers on "The Education and Training of a Civil Engineer".

Meetings at Cardiff will be held at the South Wales

Institute of Engineers, Park Place and at Swansea in the Mackworth Hotel, High Street, commencing at 6.30 p.m.

Hon. Secretary: K. J. Stewart, A.M.I.Struct.E., A.M.I.C.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

The following meetings have been arranged:

Friday, 17th *October*, 1958

Mr. Peter Gardiner, M.I.Struct.E., will give the Chairman's Address, entitled "Modular Co-ordination in Building".

Thursday 20*th November*, 1958

Joint Meeting with The Institution of Civil Engineers.

Mr. M. Holmes, B.Sc., A.M.I.C.E., on "The Strength of Lattice Girders".

Friday 5*th December*, 1958

Two or three short papers will be given by Graduates or Students of the Institution.

Meetings will be held in the Small Lecture Theatre, University Engineering Laboratories, University of Bristol, commencing at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary: A. C. Hughes, M.Eng., A.M.I.Struct.E., A.M.I.C.E., 21, Great Brockeridge, Bristol 9.

YORKSHIRE BRANCH

The following meetings have been arranged:

Wednesday, 15*th October*, 1958

At Leeds. Professor R. H. Evans, C.B.E., D.Sc., Ph.D., M.I.Struct.E., M.I.C.E., M.I.Mech.E., will give the Chairman's Address.

Thursday, 16*th October*, 1958

The Chairman's Address will be repeated at Sheffield.

Wednesday, 12*th November*, 1958

At Leeds. Mr. G. B. Godfrey, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., (Associate-Member of Council), on "Tubes and Hollow Sections".

Wednesday, 3*rd December*, 1958

At Sheffield. Mr. F. J. Samuely, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., on "Precast Concrete Structures with Particular Reference to Folded Slab Roof Construction".

Wednesday, 10*th December*, 1958

At Leeds. Mr. S. Barlow, M.I.Struct.E., and Mr. G. Foster, A.M.I.Mech.E., on "Universal Beams and their Application".

Meetings at Leeds will be held at the Metropole Hotel, King Street, and meetings at Sheffield at the Royal Victoria Hotel. Meetings will commence at 6.30 p.m., preceded by buffet tea at 6.15 p.m.

Hon. Secretary: W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorks.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary: A. E. Tait, B.Sc., A.M.I.Struct.E., A.M.I.C.E., P.O. Box 3306, Johannesburg, South Africa.

During weekdays Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone 34-1111, Ext. 257.

Natal Section Hon. Secretary: J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.

Cape Section Hon. Secretary: R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

EAST AFRICAN SECTION

Chairman: R. A. Sutcliffe, M.I.Struct.E.,

Hon. Secretary: K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Chairman: Mr. T. Karmakar, M.I.Struct.E.

Hon. Secretary: Mr. W. N. Cursitor, B.Sc., A.M.I.Struct.E., c/o Redpath Brown & Co. Ltd., P.O. Box 648, Singapore.

Acting Hon. Secretary: Mr. Chin Fung Kee, Department of Engineering, University of Malaya, Singapore 10.

ADDITIONS TO THE LIBRARY

American Concrete Institute Book of Standards, 1957 Edition. Detroit, Michigan.

American Concrete Institute. Manual of Standard Practice for Detailing Reinforced Concrete Structures. Detroit, Michigan, 1957.

American Concrete Institute. Manual of Concrete Inspection, 4th Edition. Detroit, Michigan, 1957.

Aluminium Development Association. A Symposium on Aluminium and its Alloys in Electrical Engineering. London, 1957.

American Society for Testing Materials, 1957 *Supplement to Book of A.S.T.M. Standards, including Tentatives*, Parts 1, 2, 3, 4 and 7. Philadelphia, U.S.A., 1958.

BOGUSLAVSKY, B. W. *Design of Reinforced Concrete.* New York, 1956. Presented by Mr. H. A. Hubbard.

British Wood Preserving Association. Record of the 1957 *Annual Convention.* London, 1957.

COHEN, J. S. *Tables for the Design of Beams and Slabs according to the New Code.* London, 1958.

COUSINS, H. G. *The British Standard Code of Practice for the use of Normal Reinforced Concrete in Buildings.* London, 1958.

COWAN, H. J. (Editor). *Architectural Science, No.* 2 *Structure and Architecture. A Symposium of Twelve Lectures.* Sydney University Extension Board, 1957.

CRESWELL, D. A. and KING, J. H. G. *The Structural Engineer's Data Book.* London, 1957.

CRESWELL, W. T. *The Law Relating to Building and Engineering Contracts*, 6th Edition. London, 1957.

FOX, Samuel & Co., Ltd. *Data on Creep and Heat Resisting Steels.* Sheffield.

GEDDES, J. D. *The Prediction and Measurement of Temperatures in Concrete Structures with particular reference to some recent construction in Frozen Ground.* University of Durham, King's College, Dept. of Civil Engineering Bulletin No. 11.

High Paddington Committee of Investigation. High Buildings in the United Kingdom. London, 1958. Presented by the Chairman of the Committee.

International Association for Bridge and Structural Engineering, Final Report of the 5*th Congress.* Lisbon, 1957.

International Association for Bridge and Structural Engineering Publications, Vol. XVII, 1957. Zurich, 1958.

KANI, G. *Analysis of Multistory Frames.* London, 1957.

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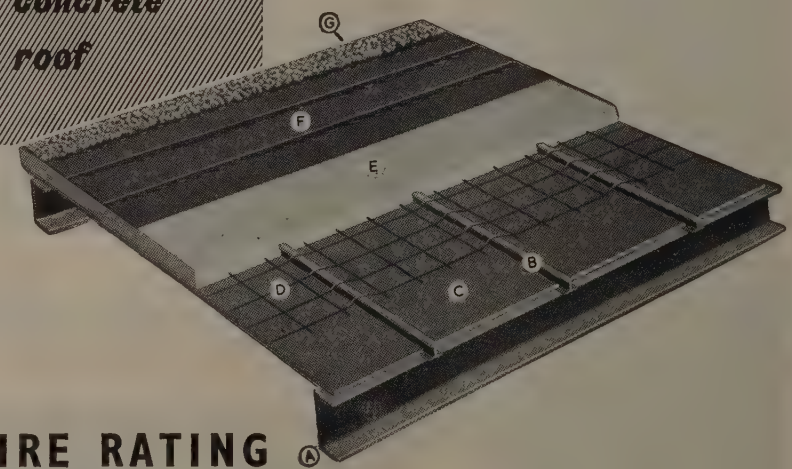

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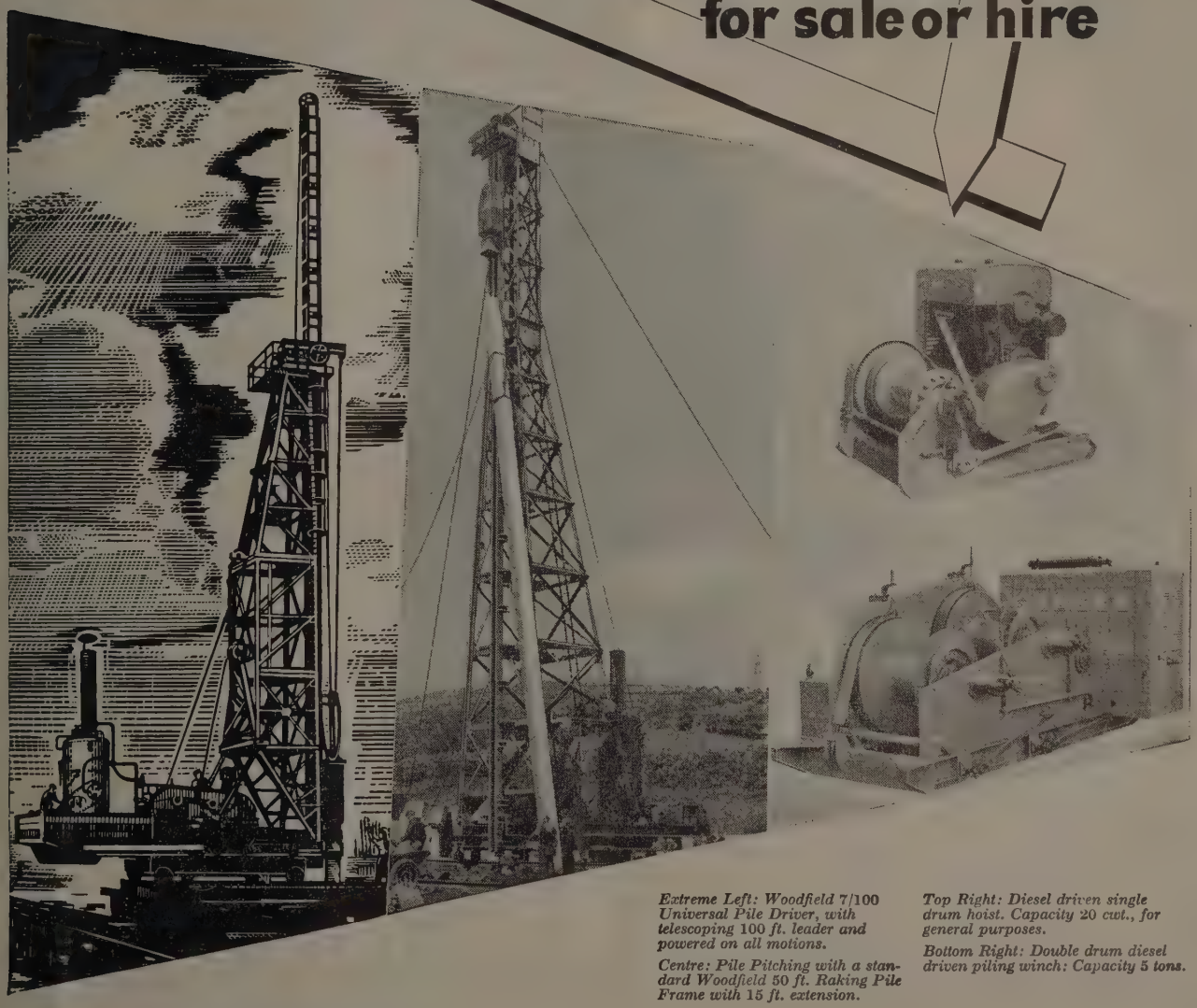

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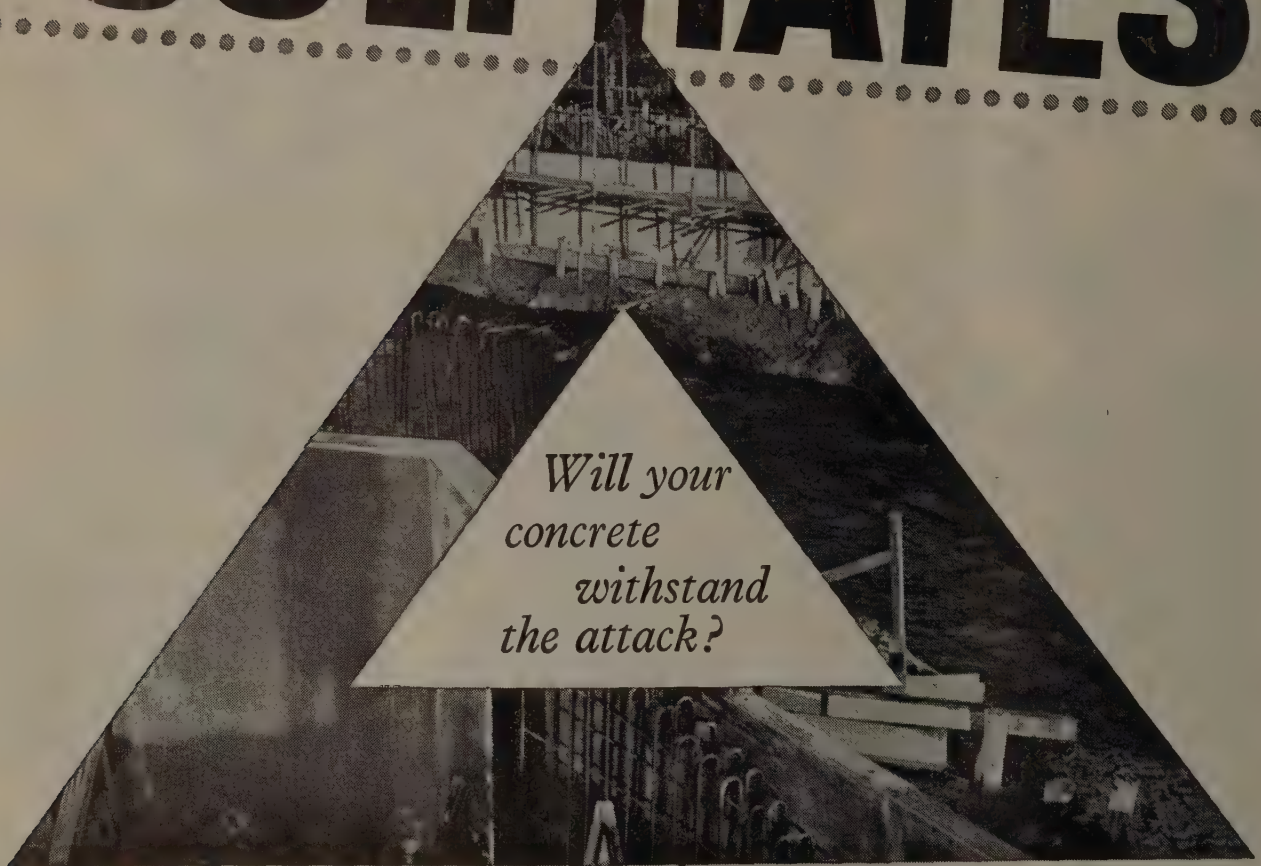

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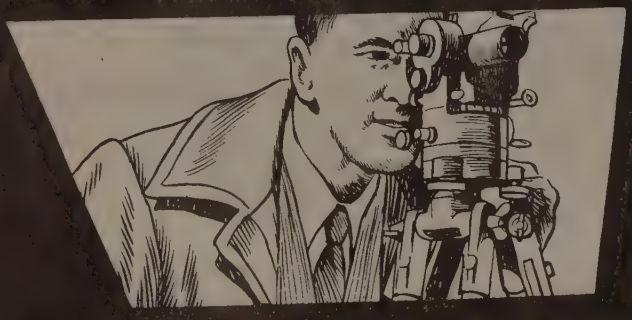

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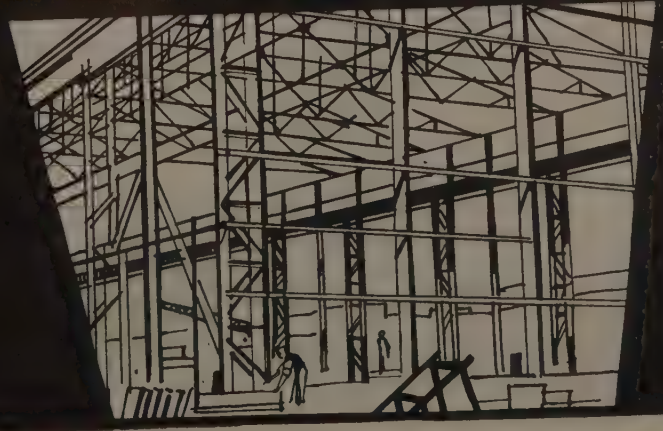

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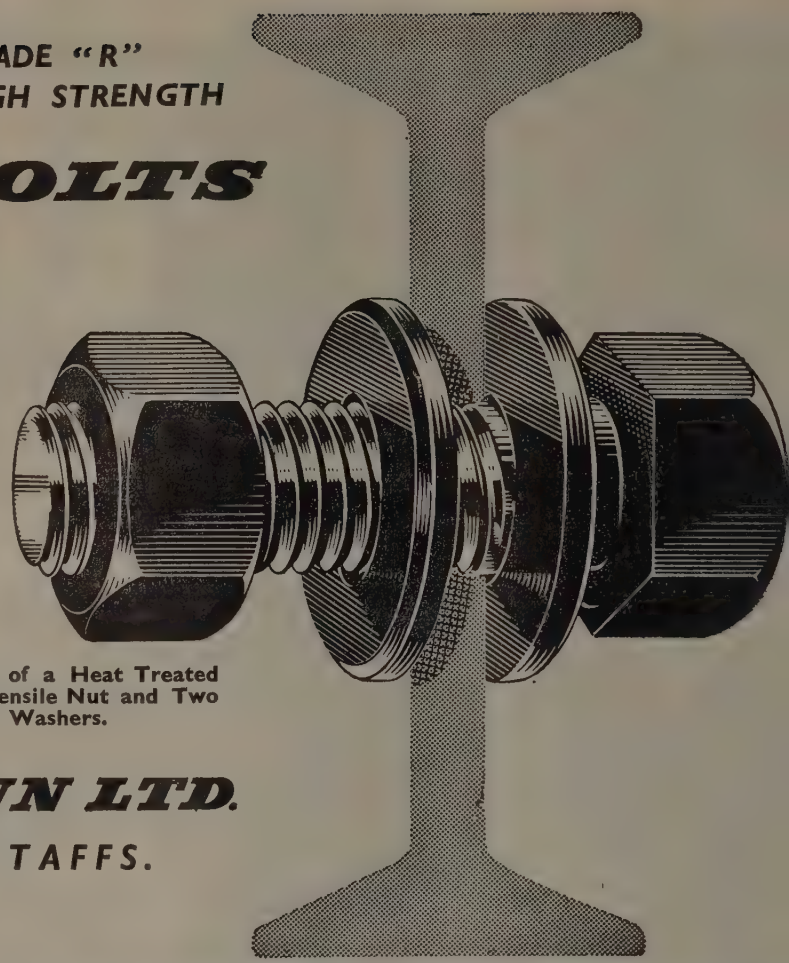

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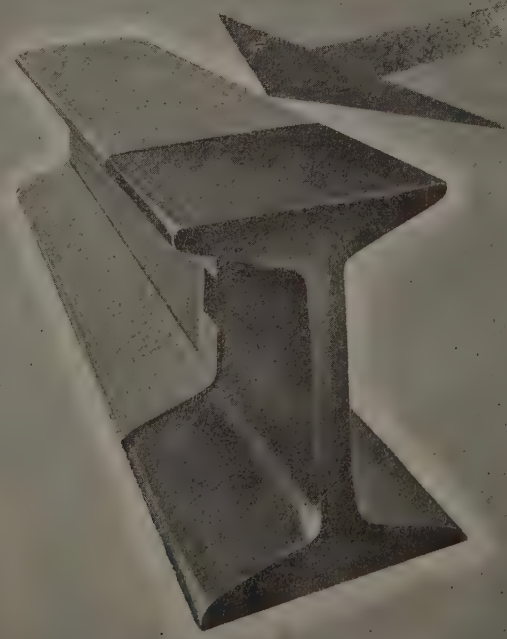

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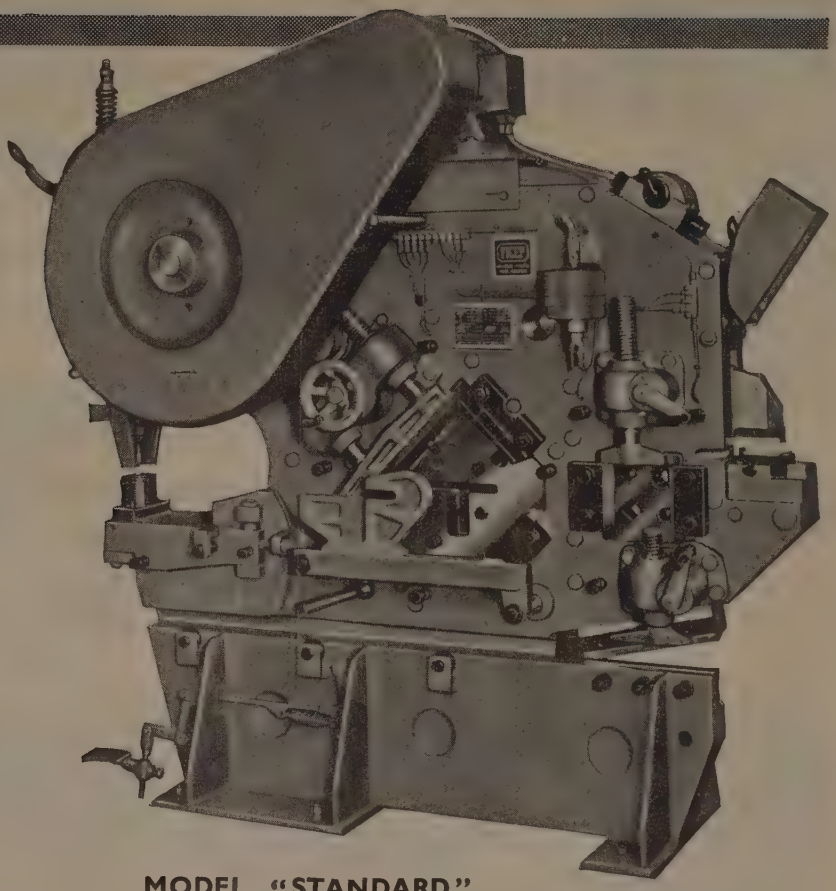

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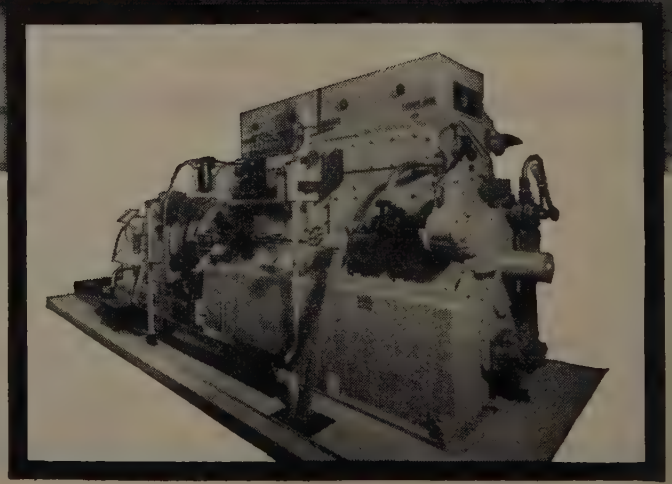

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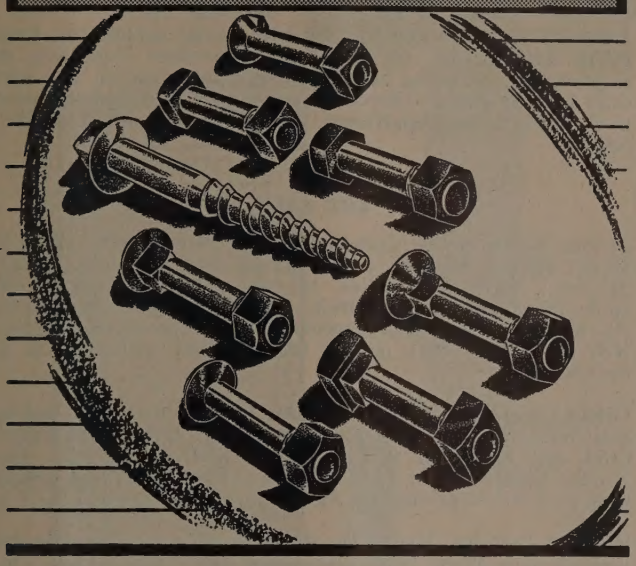

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